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SEISMIC DESIGN CRITERIA FOR BASE ISOLATED U.S. NAVY ESSENTIAL BUILDINGS

ABSTRACT This report presents design criteria for laminated rubber seismic isolation systems for use in construction of Navy essential buildings. Base isolation has been shown to limit seismic motions transmitted to structures and is effective in reducing damage. The design criteria, applicable only to structures having structural detailing as required in Seismic Zones 3 and 4, utilizes two structural load levels and has levels of performance associated with each. A structure designed according to these provisions will withstand the most probable maximum earthquake with inelastic demand ratios of 2.0 in beams and 1.25 in columns.

NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME CALIFORNIA 93043

METRIC CONVERSION FACTORS

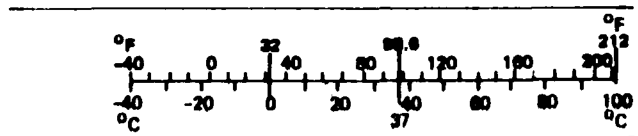
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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This report presents design criteria for laminated rubber seismic isolation systems for use in construction of Navy essential buildings. Base isloation has been shown to limit seismic motions transmitted to structures and is effective in reducing damage. A structure designed according to these provisions will withstand the most probable maximum earthquake with inelastic demand ratios of 2.0 in beams and 1.25 in columns.

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TABLE OF CONTENTS

1. EXECUTIVE SUMMARY

This document has drawn extensively upon work prepared by an Ad Hoc Committee of the Seismology Committee of the Structural Engineers Association of California (SEAOC) (Kircher, Hart and Romstad, 1989). That document was entitled "An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation" and was adopted by the State of California Building Safety Board for application to California Hospital Buildings.

The seismic design criteria developed as part of this research is based on limit state design theory and it assumes that the building can be substantially decoupled from potentially damaging earthquake motions by using base isolation. Thus the level of building response can be significantly reduced. A general discussion of seismic isolation, recent advancements and its advantages as it relates to US Navy facilities are documented elsewhere Hart, 1989.

The design criteria presented herein is only applicable to laminated layer rubber seismic isolation systems.

Acceptable performance in the context of this criteria means that the isolators will:

- remain stable for required design displacements,
- provide increasing energy dissipation with increasing displacement,
- not suffer a loss in force resisting capacity under repeated cyclic loading, and
- have quantifiable engineering parameters (e.g., force-deflection characteristics and damping).

The design criteria presented herein is only for US Navy Buildings that have structural detailing consistent with that required in Seismic Zones 3 or 4. In this Seismic Zone special structural detailing requirements exist and therefore system ductility is expected. The design criteria requires two structural analyses. The first analysis, denoted Analysis 1, is the analysis used to develop the design and it is consistent with the U.S. Navy Essential Building Criteria. This analysis can be a response spectra analysis. Alternately, the first analysis, can be a time history structural dynamic analysis of the isolated building. The second analysis, denoted Analysis 2, is a response spectra analysis that is only intended to provide a lower bound safety net design. Both of these analysis are intended to insure that the isolated structure can experience a very severe earthquake with limited ductility demands and remain stable.

The seismic design criteria utilizes in Analysis 1 two basic levels of ground motion that are denoted EQ-I and EQ-II. The EQ-I earthquake ground motion has approximately a 50% chance of being exceeded in a 50 year time period. Structural elements above the isolation system are required to remain essentially linear elastic for the EQ-I earthquake. Twenty percent of the beams in any one story can not have an inelastic demand ratio (IDR) of more than 1.25 and 10% of the columns must not have an IDR of more than 1.25. A second design earthquake, denoted EQ-II, is the most probable maximum earthquake that can be expected to take place. This design level earthquake has approximately a 10% chance of being exceeded in a 100 year time period. For the EQ-II earthquake the building can respond inelastically but must have sufficient capacity to resist system collapse. The IDR for beams and columns are limited for the EQ-2 earthquake to 2.0 and 1.25, respectively. The IDR limits noted above for EQ-II are for ductile moment resisting space frame building systems. For other systems the IDR limits are as given in Table 4-2 of the Technical Manual TM 5-810-10-1, Seismic Design Guidelines for Essential Buildings.

Analysis 2 is intended to provide a minimum force and displacement capacity safety net design. In this analysis the structural system design in Analysis 1 is assumed to be a fixed base building at the isolation level. It is then subjected to the same EQ-I earthquake response spectra as developed as part of the Analysis 1 scope of work. The beams and columns in the building when subjected to the EQ-I earthquake, at the fixed base building period, must have an IDR less than 3.

2. SEISMIC DESIGN CRITERIA

A. LATERAL-FORCE DESIGN REQUIREMENTS

A.1 General

Every seismic-isolated building and every portion thereof shall be designed and constructed to resist the deformations and forces produced by lateral displacements and forces as provided in this chapter. The deformations and forces shall be calculated as the effect of a force applied horizontally and the isolation-interface level and at each floor level or roof level above the interface level. The force shall be assumed to come from any horizontal direction.

Isolator units shall be tested to verify deformational properties and design capacity for the lateral displacements prescribed by this document.

Where prescribed wind forces produce greater deformations or stresses, such loads shall be used in lieu of forces resulting from earthquake forces.

These provisions may only be used for buildings that satisfy the following conditions:

(1) No plan or vertical irregularities exist.

(2) The elastic period of the building on the isolator system shall not exceed 3.0 sec and shall be at least four times the building period calculated assuming that the building is fixed at the isolator level.

(3) The building is designed to meet the detailing requirements of Seismic Zones 3 and 4.

(4) No significant mass eccentricities exist.

A.2 Definitions

The following definitions apply to the provisions of this section:

EQ-I EARTHQUAKE is the earthquake ground shaking corresponding to an event with approximately a 50% chance of being exceeded in a 50 year time period.

EQ-II EARTHQUAKE is the earthquake ground shaking corresponding to an event with approximately a 10% chance of being exceeded in a 100 year time period.

DESIGN DISPLACEMENT is the maximum lateral displacement required by Analysis 1.

MAXIMUM OFFSET is the residual or permanent displacement which would occur in the isolation system at the end of free-vibration response of the isolated structure.

INELASTIC DEMAND RATIO (IDR) is the ratio of the demand force to the capacity force of any structural element. This inelastic demand ratio sets the limit of the force ductility demand on the element, therefore it is element dependent as well as structural system dependent. The limiting values of the IDR's shall be obtained from Table 4-2 of the Technical Manual TM 5-810-10-1, Seismic Design Guidelines for Essential Buildings.

ISOLATION SYSTEM is the collection of structural elements which includes all individual isolator units, all structural elements which transfer force between components of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system. The force deflection characteristics of the isolation system may be characterized as linear if force remains essentially proportional to deflection, or as nonlinear for systems which either harden or soften with increased deflection or are governed predominantly by friction force.

ISOLATION INTERFACE is the boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure.

ISOLATION UNIT is a flexible structural component of the isolation system which permits large lateral deformations under design seismic load. Conceptually, isolator units may be thought of as either a spring, a damper or a frictional device, or any combination of the above; and may be used either as part of or in addition to the weight supporting system of the building.

WIND-RESTRAINT SYSTEM is the collection of structural elements which provide restraint for the seismic-isolated structure for wind loads. The wind-restraint system may be either an integral part of the isolator unit or may be a separate device.

A.3 Symbols and Notations

The following symbols and notations apply to the provisions of this section:

D = Lateral seismic displacement of the isolation system (in inches) at the center of rigidity, of the structure in the direction under consideration.

F_n = Maximum negative force in the isolation system during each cycle of testing.

F_p = Maximum positive force in the isolation system during each cycle of testing.

F_x = Lateral force applied at level x.

g = Acceleration due to gravity.

k_{eff} = Effective stiffness of isolation system determined by cyclic testing.

T = Period of seismic-isolated structure in seconds in the direction under consideration.

V_b = The total lateral force or shear on elements at or below the isolation interface.

V_s = The total lateral force or shear on elements above the isolation interface.

W = The total dead load of the structure above the isolation interface, and applicable portions of other loads.

w_i, w_x = The portion of W which is located or assigned to levels i or x, respectively.

Z_b = Numerical coefficient related to the seismicity of a region. Seismic Zone 3 and 4 that have Z_b values of 0.3 are 0.4, respectively.

Δ_p = Maximum positive displacement of isolation system during each cycle of testing.

Δ_n = Maximum negative displacement of isolation system during each cycle of testing.

K = Horizontal force factor, as per Tri-Service Manual TM 5-809-10.

B. MINIMUM EARTHQUAKE DISPLACEMENTS AND FORCES

Every seismic-isolated building shall be designed and constructed to resist minimum earthquake displacements and forces as specified by Sections C and D.

The total lateral seismic force above the isolators shall not be less than:

1. The forces resulting from Analysis 2.

2. The base shear corresponding to the design wind load and,

3. The yield level of a stiffness softening system, the ultimate capacity of a sacrificial wind-restraint system, and the static friction level of a sliding system.

If the reduced base shear force is less than these limits, then the Analysis 1 design forces shall be increased proportionately so that the greater of the three limits is satisfied.

C. ANALYSIS 1: DYNAMIC ANALYSIS

C.1 General

This section defines the minimum earthquake displacements and forces that are to be used for design.

C.2 P-eliminary Design

A preliminary isolation system design shall be developed to withstand lateral seismic displacements which act in the direction of each of the main horizontal axes of the structure in accordance with Formula (C-1).

$$D = 10\,Z_b T \tag{C-1}$$

The isolated-structure period, T, shall be determined using the deformation characteristics of the isolation system in accordance with the formula:

$$T = 2\pi\sqrt{\frac{W}{k_{eff}\,g}} \tag{C-2}$$

The value of T obtained from Formula (C-2) shall not exceed 3.0 seconds.

The isolation system, the foundation system, and all structural elements below the isolation system shall be preliminary designed to withstand a lateral seismic force, V_b, using all appropriate provisions for a non-isolated structure where:

$$V_b = \frac{k_{eff} D}{1.5} \tag{C-3}$$

The structure above the isolation system shall be preliminary designed to withstand a minimum shear force, V_s , using all appropriate provisions corresponding to the K value for a non-isolated structure where:

$$V_s = \left[\frac{k_{eff} D}{1.5}\right] K \qquad \text{(C-4)}$$

The total force shall be distributed over the height of the structure above the isolation interface in accordance with the following formula:

$$F_x = \frac{V_s \, w_x}{\sum_{i=1}^{n} w_i} \qquad \text{(C-5)}$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at the level. Forces in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

C.3 Dynamic Analyses Procedure

Base isolation design displacements and forces shall be obtained from a dynamic analysis. The analytical model of the building shall be three-dimensional and shall include both the deformational characteristics of the isolation system and the deformational characteristics of the building. An elastic Response Spectrum Analysis may be used to calculate the force in all structural elements for the EQ-I earthquake and the isolator stiffness is equal to a k_{eff} value consistent with the EQ-I isolator response. An EQ-II analysis is required to insure that the structure has sufficient ductility capacity to withstand this event. If a response spectra analysis is used then a load deflection inelastic analysis is required as described in The Technical Manual TM 5-810-10-1 to ensure that that the system capacity exceeds the EQ-II system demand. The effective stiffness and damping values of the isolation system used in the response spectrum analyses shall be substantiated by tests as specified in Section D. A Time History Analysis may be used in lieu of the response spectra analysis and it requires an analysis for at least six appropriate time histories.

C.4 Seismic Input

A properly substantiated, site-specific response spectra for the EQ-I and EQ-II earthquakes are required. When used in the analysis the earthquake time histories shall be selected from different recorded events and scaled such that their 5% damped response spectrum does not fall below the design or site specific spectrum by more than 10% at any period. For sites within 15 km of a major active fault the seismic impact must incorporate near-fault phenomena.

C.5 Interstory Drifts

Interstory drifts is defined as the ratio of the differential lateral displacement of the top and the bottom of a story to the story height. For Analysis 1, interstory drifts shall not exceed 0.005 when the isolated building is subjected to EQ-I, and 0.010 when subjected to EQ-II.

C.6 Dead Loads for the Isolated Building

Above the isolator the dead loads shall be increase by 10% to incorporate the effects of vertical accelerations on columns and long span beams.

D. ANALYSIS 2

The structure designed in Analysis 1 shall now be assumed to be fixed at the isolator level and the IDR for all beams and columns shall not exceed 3 for a response spectra analysis of the structure in this condition for the EQ-I event.

E. REQUIRED TESTS OF ISOLATION SYSTEM

E.1 General

The deformation characteristics and damping values used in the design and analysis shall be based on existing test data of the system and confirmed by the following tests on selected sample of the components prior to construction. The tests specified are for validating the properties of the base isolation system. They should not be considered as manufacturing quality control requirements.

E.2 Sequence of Tests

The following sequence of tests shall be performed on at least two components of the full size isolation system. For each test cyclic force-deflection and hysteretic behavior of the test specimen shall be recorded. The test specimens shall include the wind restraint system, if such system is used in the design. Specimens tested shall not be used for construction.

1. Twenty cycles of loading at a force corresponding to the design wind force. If a sacrificial wind-restraint system is to be utilized its ultimate capacity shall be established by test.

2. Three cycles of loading at each of the following increments of the design displacement or design force: 0.25, 0.50, 0.75, 1.0, 1.25, 1.50.

3. Ten cycles of loading at 1.50 times the design displacement.

If an isolator is also a vertical load carrying element then item 3 of the above sequence of tests shall be performed for three different vertical loads as follows:

i) DL

ii) DL + 25% LL + 1.2 (Overturning Force)

iii) DL - 25% LL - 1.2 (Overturning Force)

E.3 Determination of Stiffness Characteristics

The effective stiffness of the system at each test displacement shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n} \tag{E-1}$$

where F_p, Δ_p and F_n, Δ_n are the maximum positive and negative forces and displacements, respectively.

E.4 Determination of Damping Characteristics

The equivalent viscous damping ratio (β) for each cycle of loading shall be calculated as:

$$\beta = \frac{1}{2\pi}\left[\frac{\textit{Area of Hysteresis Loop}}{k_{eff}\ \Delta_{\max}^2}\right] \tag{E-2}$$

E.5 System Adequacy

The base isolation system test performance shall be assessed as adequate if:

1. The measured force deflection relationships for all tests specified in Section E.2 has an increase in maximum positive and negative force for all increases in corresponding maximum displacements.

2. There is less than a 10% change in amplitude of zero to maximum positive or negative force, effective stiffness and equivalent viscous damping for any of the cycles of test performed at a given displacement level as specified in Section E.2.

F. ADDITIONAL REQUIREMENTS

F.1 Environmental Conditions

In addition, to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

F.2 Wind Loads

Isolated structures shall resist design wind loads at all levels above the isolator level in accordance with the general wind design provisions. At the isolator level a wind restraint system shall be provided to limit story drift to 0.002 times the story height.

F.3 Fire Resistance

Fire resistance for the isolation system shall meet that required for the building's columns, walls or other structural elements.

F.4 Lateral Restoring Force

The isolation system shall be configured to produce a restoring force sufficient to ensure that the maximum offset of any isolator unit does not exceed 15% of the design displacement calculated in Sections B and C.

The maximum offset of any isolator unit shall be determined by an experimental snap-back test or by an analytical free-vibration time history analysis. If time history analysis is used to determine maximum offset, then the hysteretic behavior of the isolation system shall be modeled explicitly using the force-deflection characteristics determined by Section E.2 tests. For either snap-back testing or time history analysis, mass shall be included, as necessary, to accurately represent the inertial effects of the isolated structure.

F.5 Vertical Load Stability

The isolation system shall provide a factor of safety of three (3) for vertical loads (dead loads plus live load) in its laterally undeformed state. It shall also be designed to be stable under the full design vertical loads at a horizontal displacement which is the greater of either 1.5 times the design displacement or four times the maximum offset for softening systems and sliding systems, or 1.5 times the design force for hardening systems.

F.6 Overturning

The factor of safety against global structural overturning at the isolation level shall be not less than 1.0 for the load combinations required by US Navy Criteria TM 5-810-10-1. All gravity and seismic loading conditions shall be investigated, except that seismic forces for overturning calculations shall be based on the maximum base shear force for the superstructure; and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of building elements.

F.7 Inspection and Replacement

Access for inspection and replacement of the isolation system shall be provided.

F.8 Quality Control

A quality control testing program for the isolation system shall be established by the design engineer.

F.9 Design Review

Engineering review of the isolation system concept and design is required.

F.10 Lateral Drift of the Structural System

The structure above the isolation system shall conform to the drift criteria in the US Navy Essential Building criteria for EQ-I and EQ-II earthquakes.

F.11 Separations

Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall be not less than the greater of either 1.5 times the design displacement, four times the maximum offset or the minimum distance required for conventional structures. The intent of this provision is to avoid impact with adjacent structures.

APPENDIX A

REFERENCES

ATC, 1978 *Tentative Provisions for the Development of Seismic Regulations for Building*, Applied Technology Council Report ATC-3-06, Redwood City, California.

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TM 5-809-10, 1986, *Seismic Design for Buildings, Basic Design Manual*, Departments of the Army, the Navy and the Air Force Report TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13, Washington, D.C.

Teal, E.J., *Seismic Drift Control Criteria*, AISC Engineering Journal Vol. 12, No. 2, 1975.

APPENDIX B

CASE STUDY BUILDING - NO ISOLATORS

B.1 DESCRIPTION OF BUILDING

A three-story building is considered in this appendix. The building is a modified version of a building example discussed in Technical Manual TM 5-810-10-1. The lateral seismic resistance is provided by transverse ductile moment-resisting steel frames (DMRSF) and longitudinal steel braced frames (BSF). There are a series of interior vertical load-carrying columns and girders in addition to the space frame, see Figure (B.1-1).

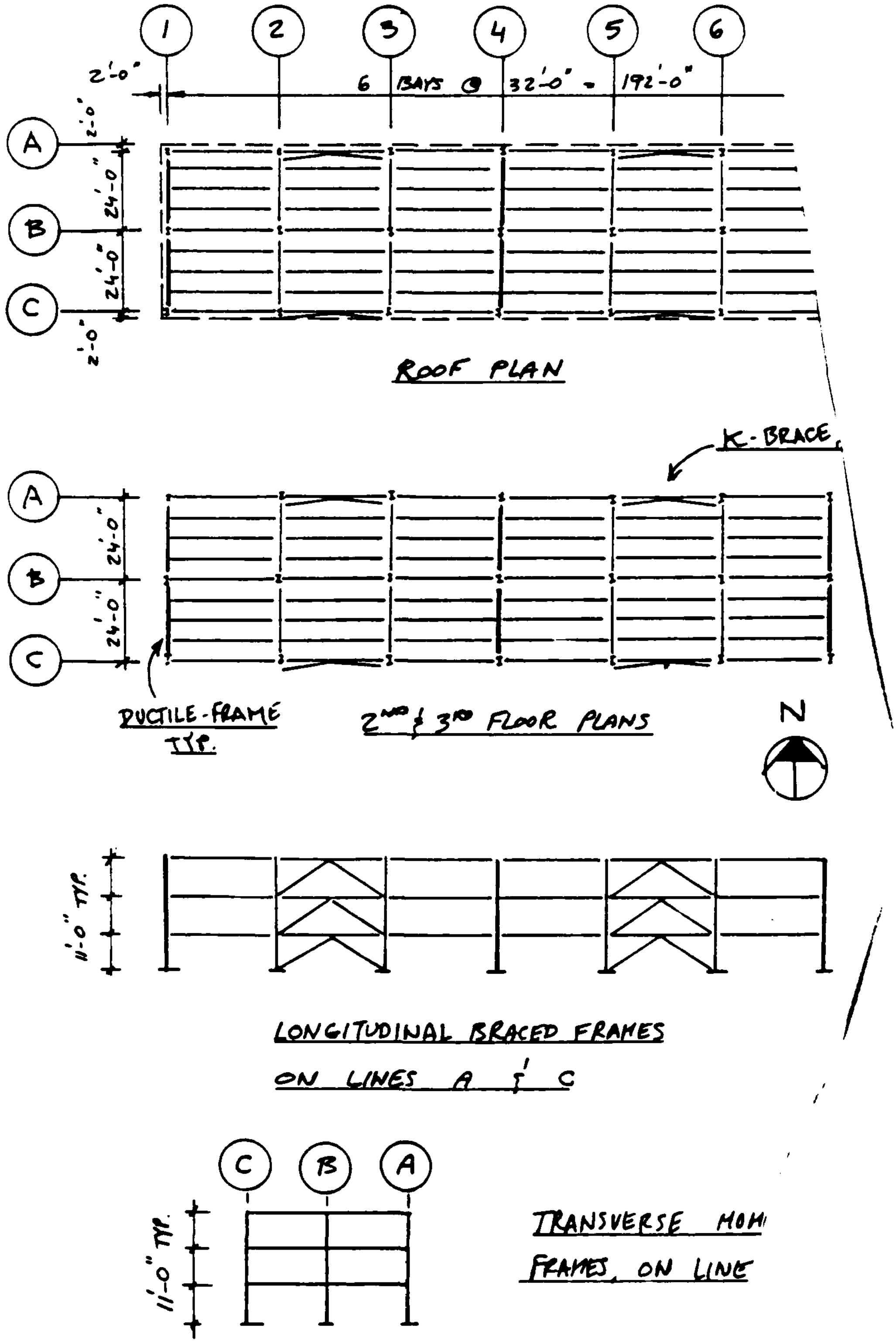
The building is considered to be located in Seismic Zone No. 4 and the soil profile of the site consists of a dense soil where the depth exceeds 200 feet. Table B.1-1 shows roof and typical floor loading. Only the transverse direction is considered in this case study.

B.2 DESIGN USING UBC-85

B.2.1 General

The base shear coefficient is calculated using the 1985 Uniform Building Code (UBC) procedure, and then the resulting base shear in each direction is distributed to the stories. The roof and the floor systems form rigid diaphragms. Therefore, the story forces are distributed to the frames in direct relation to each frame stiffness.

The stresses in beams are checked using working stress criteria. While columns stresses are checked using both working stress and ultimate stress (strength) approaches.



1
2
3
4
5
6
2'-0"
6 BAYS @ 32'-0" = 192'-0"
A
B
C
2'-0"
24'-0"
24'-0"
2'-0"
ROOF PLAN
K-BRACE
A
B
C
24'-0"
24'-0"
DUCTILE-FRAME TYP.
2ND & 3RD FLOOR PLANS
N
11'-0" TYP.
LONGITUDINAL BRACED FRAMES ON LINES A & C
C
B
A
11'-0" TYP.
TRANSVERSE MOM
FRAMES, ON LINE

Figure B.1-1

Table B.1-1

Loads:	Roof: DL = 25.7 psf, Floor: DL = 74.5 psf,	LL = 20.0 psf LL = 50.0 psf
Weights:	Roof total weight Floor total weight Total building weight	= 375 kips = 707 kips = 1789 kips

Table B.2-1

Seismic Zone Coefficient (zone 4)	Z = 1.0
Occupancy Importance Factor (essential building)	I = 1.5
Horizontal Force Factor (DMRSF, BSF)	K = 0.67
Soil Profile Coefficient (soil type S2)	S = 1.2
Building Period T	T = 0.67 sec.
Numerical Coefficient C $C = 1 / 15\sqrt{T} < 0.12$	C = 0.081
CS < 0.140	CS = 0.097
Base Shear Coefficient	ZIKCS = 0.098
Base Shear V = ZIKCS W	V = 175 kips

B.2.2 Building Period

1. The period is calculated using the 1985 UBC, Chapter 23, Section 2312, Formula 12-3B: T = 0.1 N where N is the number of stories. It follows that T = 0.1 (3) = 0.3 seconds. This period is conservative and provides a lower bound.

2. Alternate Method

   a. *Upper Bound*: The period is calculated using the 1985 UBC, Chapter 23, Section 2312, Formula 12-3.

$$T = 2\pi \sqrt{\frac{\Sigma\, W_i \delta_i^2}{g\, \Sigma\, f_i\, \delta_i}}$$

Or using Teal's Formula (AISC Engineering Journals, 2nd and 4th Quarters, 1975), which is its equivalent.

$$T = 0.25 \sqrt{\frac{\Delta}{C_1}} = 0.25 \sqrt{\frac{\text{Roof deflection}}{\text{Roof acceleration}}}$$

In this later formula, the following assumptions are made:

$$\Delta \approx \frac{2}{3}\,(\text{allowable deflection}) = \frac{2}{3}\,(0.005\; h_n)$$

$$C_1 \approx ZICS \quad \text{where} \quad C = \frac{1}{15\sqrt{T}}$$

Substituting these values of Δ and C_1 into Teal's formula:

$$T = 0.11 \left[\frac{h_n}{ZIS}\right]^{\frac{2}{3}}$$

where h_n is the total height of the building (feet). It follows that:

$$T = 0.11 \left[\frac{33}{1 \times 1.5 \times 1.2}\right]^{\frac{2}{3}} = 0.76 \quad \text{seconds}$$

b. *Limiting Value:* Use the Navy's Design Basic Manual Formula (Chapt. 4, Para. 4-3d(5))

$$T = 1.4\, C_T h_n^{\frac{3}{4}} \quad \text{where} \quad C_T = 0.035 \ \text{for steel frame}$$

$$T = 1.4\,(0.035)\,(33)^{\frac{3}{4}} = 0.67 \ \text{seconds}$$

B.2.3 Design Force

The base shear design force is determined for the transverse direction in Table B.2-1 following the 1985 UBC, Chapter 23, Section 2312 procedure.

B.2.4 Distribution of Forces to Frames

The resulting base shear is distributed to the roof and floors to obtain the story forces as shown in Table B.2-2. Then the story forces are distributed to the frames based upon their relative rigidity. In the transverse direction, there are three identical moment resisting frames and in the longitudinal direction there are two K-braced frames, Figure B.1-1.

Because of symmetry there is no "calculated" torsion. However, the "accidental" torsion is the story force times the nominal eccentricity of 5% of the maximum building dimension. The resulting torsional shear is distributed to each frame based on the relative rigidities of the frames, see Table B.2-3.

These total forces, i.e. direct shear plus torsional shear, are then used to size the frames using the Portal Method. Then the preliminary sizes are checked to see if they meet the stress and drift limits of the 1985 UBC using a static force computer analysis.

B.2.5 Checking Transverse Direction, Frame 4

Force distribution to the frame 4

Roof	:	$0.35 \times 58 = 20$ kips
3rd floor	:	$0.43 \times 58 = 25$ kips
2nd floor	:	$0.22 \times 58 = 13$ kips

Figure B.2-1(a) shows the frame model that was used in the static analysis to determine beams and columns internal forces. Figure B.2-1(b) shows the computer results of the analysis. The moments, shears, and axial forces are shown for a load combination 1/1.33 (DL + LL + E).

The stress check for beams are shown in Table B.2-4. The working stress approach is used to check the beams where the elastic moment capacity (M_c) is compared against the moment demand (M_d) using AISC specification. Beams are Grade 36.

The stress check for columns are performed using two approaches: working stress method and strength method.

A. Elastic Approach: (working stress method)

from computer analysis: exterior column	$P = 83$	kips
	$M = 1108$	kip-in
interior column	$P = 147$	kips
	$M = 1385$	kip-in

Stability equation AISC spec. (1.6-1a) $$\frac{f_a}{F_a} + \frac{C_{max} \; f_{bx}}{\left[- \dfrac{f_a}{F'_{ex}} \right]} < 1$$

Stress equation AISC spec. (1.6-1b) $$\frac{f_a}{0.6\,F_a} + \frac{f_{bx}}{F_{bx}} < 1$$

Table B.2-2, Transverse Direction
Distribution of Base Shear to Stories

Floor Level	hx ft.	Wx K.	Wx.hx Kip-ft	$\frac{W_x h_x}{\sum W_x h_x}$	Fx K.	Vx K.	DOTM Kip-ft	OTM Kip-ft
R	33	375	12,375	0.35	61			
3	22	707	15,554	0.43	75	61	671	671
2	11	707	7,777	0.22	39	136	1,496	2,167
1						175	1,925	4,092
$\sum$		1,789	35,706	1.00	175			

Table B.2-3, Total Force per Frame

Frame	Direct Shear	Torisonal Shear	Design Shear	Force per frame (Kips)
1	0.33	0.03	0.36 Ft	$0.36 \times 175 = 63$
4	0.33	0.00	0.33 Ft	$0.33 \times 175 = 58$
7	0.33	0.03	0.36 Ft	$0.36 \times 175 = 63$

Ft, the base shear force in the transverse direction

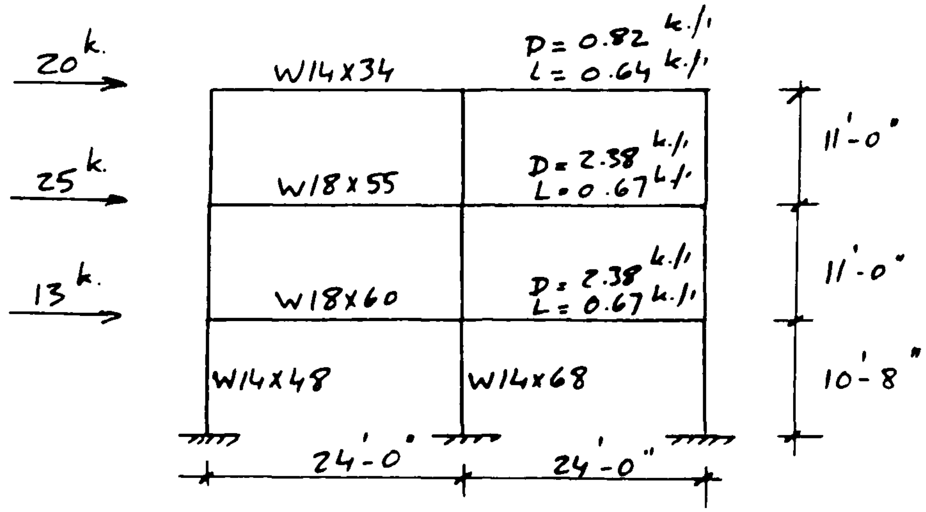


Figure B.2-1(a)

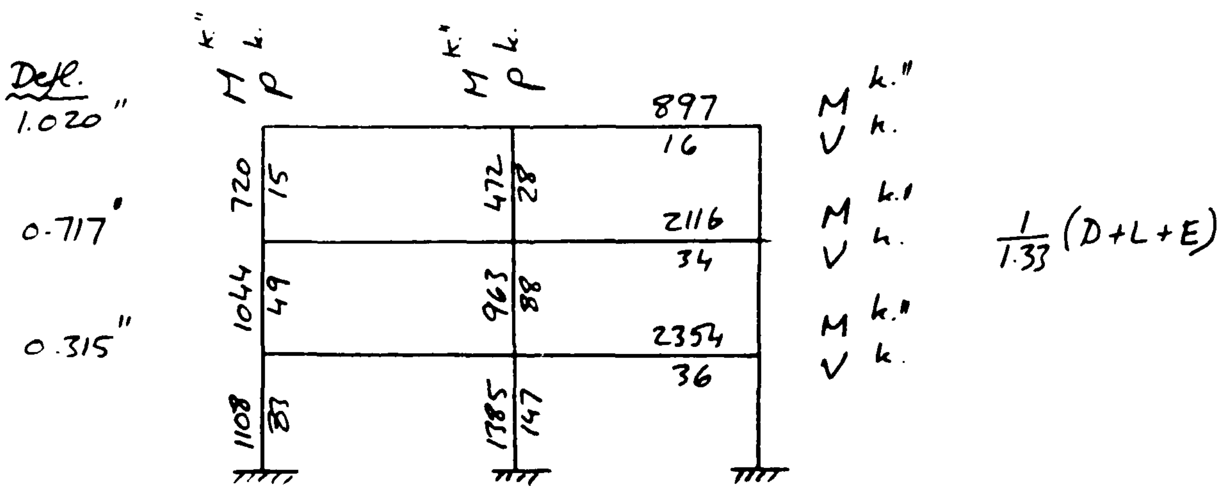
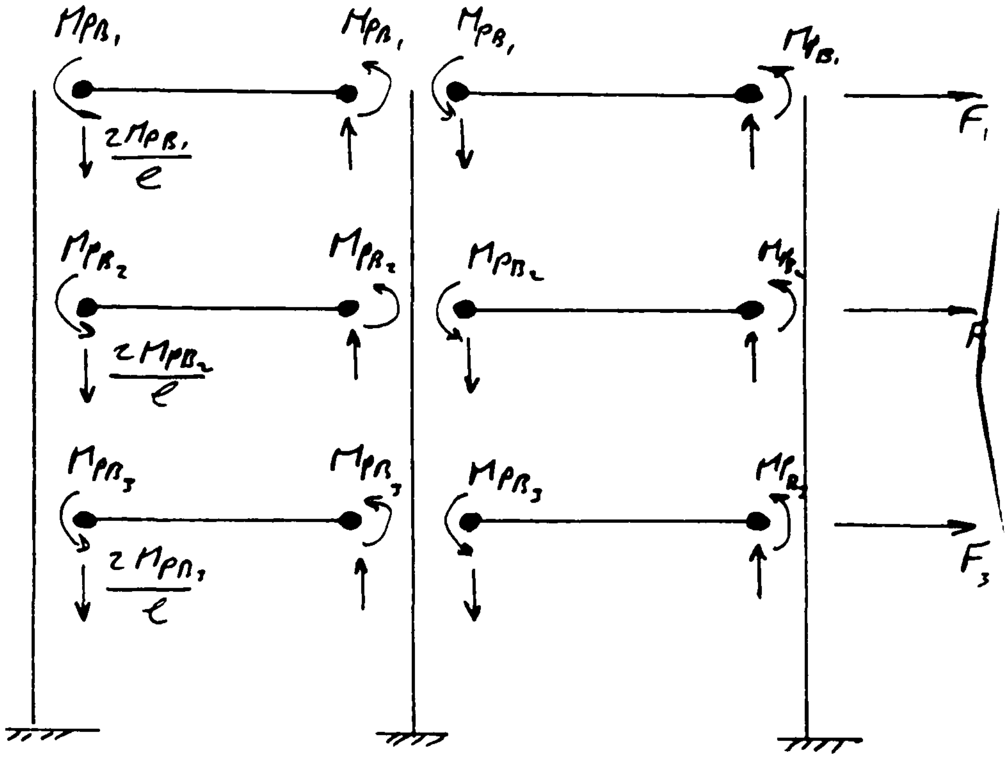
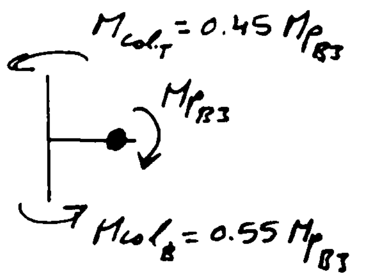


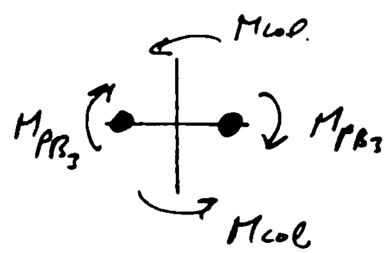
Figure B.2-1(b)



Free body diagram showing plastic hinge formation in all beams under ultimate loading conditions.



Exterior col.
1st level



Interior col.
1st level

$$P_{TOTAL} = 1.3\, P_{D+L} + \Sigma \frac{2M_{PB}}{\ell} \qquad P_{TOTAL} = 1.3\, P_{D+L}$$

$$M_{TOTAL} = 0.55\, M_{PB_3} \qquad M_{TOTAL} = M_{col} = M_{PB_3}$$

Figure B.2-2

B. Strength Approach:

criteria: -strong columns weak beam frame
-assume plastic hinge formation in all beams while columns remain elastic

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W14 × 34	1965	14
3	W18 × 55	4032	28
2	W18 × 60	4428	31

			73

-Exterior Column

(Free body diagram, see Figure B.2-2)

$M = 0.55\ M_p\ (\text{beam}) = 0.55 \times 4428 = 2436$ kip-in

$P = 1.3\ (D+L) + 2M_p/L = 1.3\ (67+23) + 73 = 165$ kips

(AISC specifications, section 2.1)

$M = 1.3\ (D+L+E) =$ (computer results) $= 1916$ kip-in

$P = 1.3\ (D+L+E) =$ (computer results) $= 144$ kips

- Interior Column

(Free body diagram, see Figure B.2-2)

$M = 1.0\ M_p\ (\text{beam}) = 1.0 \times 4428 = 4428$ kip-in

$P = 1.3\ (D+L) + 2M_p/L = 1.3\ (46+50) = 255$ kips

(AISC) specifications, section 2.1)

$M = 1.3\ (D+L+E) =$ (computer results) $= 2395$ kip-in

$P = 1.3\ (D+L+E) =$ (computer results) $= 255$ kips

-Design forces: exterior column $M = 2436$ kip-in
$P = 165$ kips

interior column $M = 4428$ kip-in
$P = 255$ kips

Stability equation AISC spec. (2.4-2)

$$\frac{P}{P_{cr}} + \frac{C_{mx} M}{\left(1 - \frac{P}{P_e}\right) M_m} < 1$$

Stress equation AISC spec. (2.4-3) $\frac{P}{P_y} + \frac{M}{1.18\ M_p} < 1$

Table B.2-5 shows comparison between the allowable drift ratio and the drift ratios obtained from the computer analysis, using the 1985 UBC, Chapter 23, Section 2312(b) criteria.

Table B.2-4, Beams Check

Level	Section	Sx qu.in	Ln in.	Lc in.	Fbx ksi	Mc kip-in	Md kip-in
R	W14 × 34	54.6	6.0	7.1	24	1310	897
3	W18 × 55	112.0	6.0	7.9	24	2688	2116
2	W18 × 60	123.0	6.0	8.0	24	2952	2354

Table B.2-5, Columns Check

	Description		Exterior Column Level 1	Interior Column Level 1
	Column section	GR 50	W14 × 48	W14 × 68
P	Axial Load (working stress check)	kips	83	148
M	Moment (working stress check)	kip-in.	1108	1385
P	for strength check	kips	165	255
M	for strength check	kips.in.	2436	4428
A	cross sectional area	in.	14.10	20.00
S_x	section modulus	in3	70.30	103.00
Z_x	plastic modulus	in3	78.40	115.00
r_x	radius of gyration	in.	5.85	6.01
r_y	radius of gyration	in.	1.91	2.46
I_x	moment of intertia	in4	485	723
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.21	1.87
K_x			1.47	1.43
L unbraced length		in.	119	119
(KL/r)x			29.90	28.31
(KL/r)y			62.30	48.37
fa		ksi	5.89	7.40
fbx		ksi	15.76	13.45
Fa		ksi	22.32	24.60
Fbx		ksi	30.00	33.00
F'ex		ksi	167.01	186.27
fa/Fa			0.26	0.30
fbx/Fbx			0.53	0.41
Cmx			0.85	0.85
Cmx * fbx/(1 - fa/F'ex)/Fbx			0.46	0.46
fa/Fa + Cmx * fbx/(1 - fa/F'ex)/Fbx			0.73	0.66
fa/0.6F_y + fbx/Fbx			0.72	0.65
Strength Approach:				
Pcr		kips	535.02	836.47
P_y		kips	705.00	1000.00
P_e		kips	4513.36	7140.21
M_p		kips-in	3920.00	5750.00
Mm/M_p			0.93	1.00
P/Pcr + Cmx * M/(1 - P/Pe)/Mm			0.90	0.98
P/P_y + M/1.18/M_p			0.76	0.91

B.3 DESIGN USING UBC-88

B.3.1 General

The base shear coefficient is calculated using the 1988 Uniform Building Code (UBC) procedure, and the resulting base shear in each direction is distributed to the stories. Then the story forces are distributed to the frames in direct relation to each frame stiffness. The stresses in beams are checked using working stress criteria. While column stresses are checked using both working stress and ultimate stress (strength) approaches.

B.3.2 Base Shear

The base shear equation of UBC-88, Chapter 23, Section 2312(e)2 is $V = \frac{ZIC}{R_w} W$

where

Z	=	0.40	Seismic Zone Factor (Zone 4)
I	=	1.25	Importance Factor (essential facility)
R_w	=	12	Structural System Coefficient (SMRSF)
S	=	1.2	Site Coefficient (Soil profile S2)
C	=	$\frac{1.25}{T^{2/3}}S$	Numerical Coefficient C
			C need not be greater than 2.75
			C/R_w must be greater than 0.075

$$\Rightarrow \; C = \frac{1.25}{T^{2/3}}(1.2) = \frac{1.5}{T^{2/3}}$$

$\Rightarrow \; V = 0.0417\ CW$ the base shear formula

Building period

1. *Method A:* The period is calculated using UBC-88, Chapter 23, Section 2312, Equation 12-3

$$T = C_t (h_n)^{3/4}$$

where

$$C_t = 0.035$$

$$h_n = 33 \text{ ft.}$$

$$T = 0.035\,(33)^{3/4} = 0.48 \text{ sec.}$$

Check limits:

$$C_A = \frac{1.5}{(0.48)^{2/3}} = 2.44 < 2.75 \qquad \text{OK}$$

$$\frac{C_A}{R_W} = \frac{2.44}{12} = 0.203 > 0.075 \qquad \text{OK}$$

This method is conservative, upper bound.

2. *Method B:* The period is calculated using Equation 12-5.

$$T = 2\pi \sqrt{\frac{\Sigma\, W_i \delta_i^2}{g\, \Sigma\, f_i \delta_i}} \approx 0.25 \sqrt{\frac{\Delta}{C_1}}$$

Teal's formula (AISC Engineering Journals, 2nd and 4th Quarters, 1975).

$$\Delta \approx \frac{2}{3}\,\Delta_{allow} = \frac{2}{3}\left[\frac{0.04}{R_w}\right] h_n$$

where Δ is the deflection at roof

h_n Total building height

$$\Delta = \frac{2}{3}\left(\frac{0.04}{12}\right)(33 \times 12) = 0.88 \text{ inches}$$

$$C_1 = \frac{V}{W} = \frac{ZIC}{R_W} = \frac{0.04\,(1.25)}{12}\left[\frac{1.5}{T^{2/3}}\right] = \frac{0.0625}{T^{2/3}}$$

Substituting Δ and C_1 values in Teal's formula, it follows that

$$T = 0.25 \sqrt{\frac{0.88\ T^{2/3}}{0.0625}} \qquad \text{and}$$

$$T = 0.91 \quad \text{seconds} \qquad \text{then}$$

$$C_B = \frac{1.5}{(0.91)^{2/3}} = 1.6 \quad < \quad 2.75 \quad \text{ok}$$

$$\frac{C_B}{R_W} = \frac{1.6}{12} = 0.133 \quad > \quad 0.075 \quad \text{ok}$$

For stress calculation: The value of C shall be not less than 80 percent of the value obtained by using T from Method A.

$$C = 0.80\ C_A = 0.80\ (2.44) = 1.95$$

For drift calculation: The design lateral forces used to determine the calculated drift may be derived from a value of C based on the period derived from Method B neglecting the 80 percent limitation. Section 2312 (e) 8.

use $\quad C_B = 1.60$

Base Shear

$V = 0.0417\ C\ W$

$V\ (\text{drift}) = 0.0417\ (1.60)\ W = 0.0667\ W$

$V\ (\text{stress}) = 0.0417\ (1.95)\ W = 0.0813\ W$

ratio $V(\text{stress}) / V\ (\text{drift}) = 0.0813/0.0667 = 1.22$

$V(\text{stress}) = 0.0813 \times 1789 = 145$ kips

Distribution of Base Shear to Frames

The resulting base shear is distributed to the roof and floors to obtain the story forces as shown in Tables B.3-1(a). The story forces are distributed to the frames based upon their relative rigidity. Then the "accidental" torsional shear is added to each frame force, see Table B.3-1(b).

These total forces, i.e. direct shear plus torsional shear, are then used to size the frames using the Portal Method. Then the preliminary sizes are checked to see if they meet the stress and drift limits of the 1988 UBC using a static force computer analysis.

B.3.3 Checking Transverse Direction, Frame 4

Figure B.3-1(a) shows the frame model that was used in the static analysis to determine beam and columns internal forces. Figure B.3-1(b) shows the computer results of the analysis. The moments, shears, and axial forces are shown for a load combination 1/1.33 (DL + LL + E).

Checking Building Period

For stress analysis, we used coefficient C = 1.95. That corresponds to a period of

$$T = \left(\frac{1.5}{C}\right)^{3/2} = 0.67 \text{ sec.}$$

but from the results of the analysis: Equation 12-5 UBC-88,

$$T = 2\pi \sqrt{\frac{375 \times 0.947^2 + 707 \times 0.660^2 + 707 \times 0.289^2}{3 \times 386.4\ (17 \times 0.947 + 21 \times 0.660 + 11 \times 289)}} = 0.85 \text{ sec.}$$

Table B.2-6 Drift Check, Transverse Direction

Level	Deflection inches	Drift inches	Drift Ratio (Drift × 1/K)/h	Allowable Drift Ratio
R	1.020	0.303	0.0035	0.005
3	0.717	0.402	0.0045	0.005
2	0.313	0.315	0.0038	0.005

Table B.3-1(a) Transverse Direction
Distribution of Base Shear to Stories

Floor Level	h_x ft.	W_x K.	$W_x \cdot h_x$ Kip.ft	$\frac{W_x h_x}{\varepsilon W_x h_x}$	F_x K.	V_x K.	DOTM Kip.ft	OTM Kip.ft
R	33	375	12,375	0.35	51			
3	22	707	15,554	0.43	62	51	561	561
2	11	707	7,777	0.22	32	113	1,243	1,804
1						145	1,595	3,399
		1,789	35,706	1.00	145			

Table B.3-1(b) Total Force per Frame

Floor Level	Story Force	Frame 1	Frame 4	Frame 7
R	51	0.36 × 51 = 19	0.33 × 51 = 17	0.36 × 51 = 19
3	62	0.36 × 62 = 23	0.33 × 62 = 21	0.36 × 62 = 23
2	32	0.36 × 32 = 12	0.33 × 32 = 11	0.36 × 32 = 12

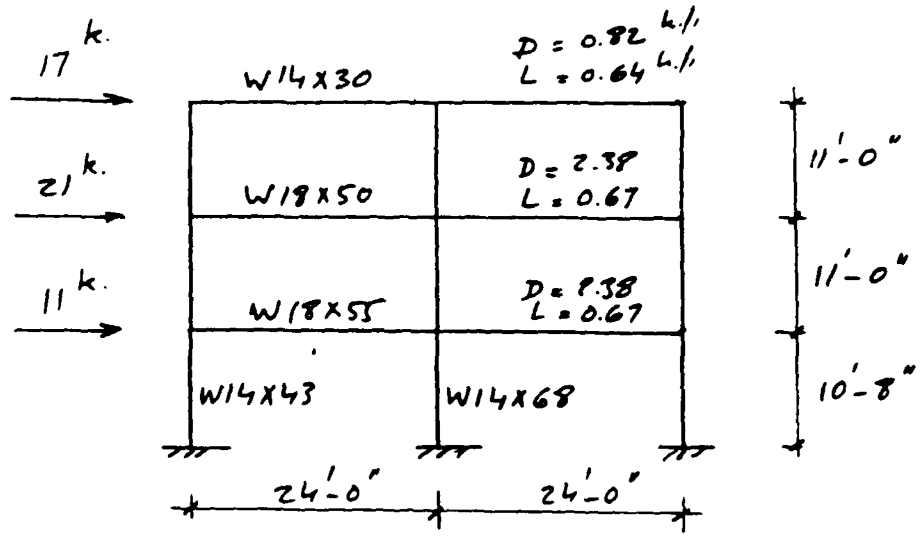


Figure B.3-1(a)

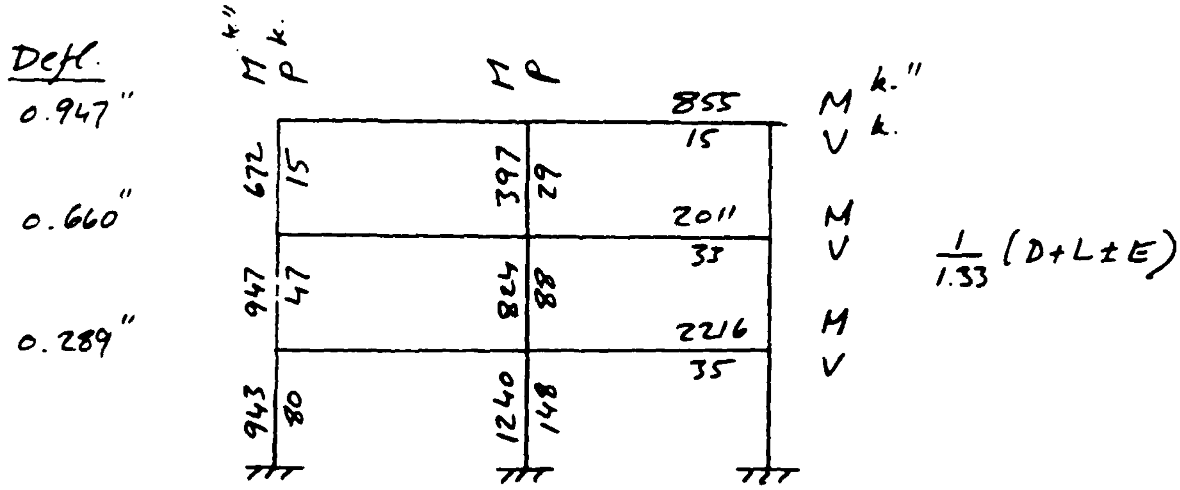


Figure B.3-1(b)

So the period assumption was conservative for stress analysis. However, using T = 0.91 sec. for drift calculations is not valid. Therefore, the period of T = 0.85 sec. is used for drift calculation:

$$C(\text{drift}) = \frac{1.5}{(0.85)^{2/3}} = 1.67$$

$$V(\text{stress})/V(\text{drift}) = 1.95/1.67 = 1.17$$

Stress Check

The stress check for beams are shown in Table B.3-2. The working stress approach is used to check the beams where the elastic moment capacity (M_c) is compared against the moment demand (M_d) using AISC specification. Beams are Grade 36.

The stress check for columns is performed using two approaches: working stress method and strength method.

A. **Elastic Approach**: (working stress method)

from computer analysis:	exterior column	P = 80	Kips
		M = 943	Kip-in.
	interior column	P = 147	Kips
		M = 1240	Kip-in

Stability equation AISC spec. (1.6-1a) $$\frac{f_a}{F_a} + \frac{C_{max} \quad f_{bx}}{(1 - f_a/F'_{ex})\, F_{bx}} < 1.0$$

Stress equation AISC spec. (1.6-1b) $$\frac{f_a}{0.6\, F_a} + \frac{f_{bx}}{F_{bx}} < 1.0$$

B. Strength Approach:

criteria:
- strong columns weak beam frame
- assume plastic hinge formation in all beams while columns remain elastic.

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W14 × 30	1702	12
3	W18 × 50	3636	25
2	W18 × 55	4032	28

			65

- Exterior Column;

(Free body diagram, see Figure B.2-2)

M	=	$0.55\ M_p$ (beam	=	0.55 × 4032	=	2218 kip-in
P	=	$1.3\ (D+L) + 2M_p/L$	=	1.3 (67 + 23) + 65	=	157 kips

(AISC specifications, section 2.1)

M	=	1.3 (D+L+E)	=	(computer results)	=	1640 kip-in
P	=	1.3 (D+L+E)	=	(computer results)	=	139 kips

- Interior Column

(Free body diagram, see Figure B.2-2)

M = $1.0\ M_p$(beam)

P = $1.3\ (D+L) + 2M_p\ /L$

(AISC specifications, section 2.1)

M	=	1.3 (D+L+E)	=	(computer results)	=	2145 kip-in
P	=	1.3 (D+L+E)	=	(computer results)	=	255 kips

-Design forces:	exterior column	M	=	2218	kip-in
		P	=	157	kips
	interior column	M	=	4032	kips-in
		P	=	255	kips

Stability equation $$\frac{P}{P_u} + \frac{C_{mx}\,M}{(1 - \frac{P}{P_e})\,M_m} < 1$$

AISC spec. (2.4-2)

Stress equation $$\frac{P}{P_y} + \frac{M}{1.18\ M_p} < 1$$

AISC spec. (2.4-3)

Table B.3-3 shows the results of the column check.

Table B.3-2, Beams Check

Level	Section	Sx qu.in	Ln in.	Lc in.	Fbx ksi	Mc kip-in	Md kip-in
R	W14 × 30	42.0	6.0	7.1	24	1008	855
3	W18 × 50	88.9	6.0	7.9	24	2134	2011
2	W18 × 55	98.3	6.0	7.9	24	2360	2216

Table B.3-3, Columns Check

	Description		Exterior Column Level 1	Interior Column Level 1
	Column section	GR 50	W14 × 43	W14 × 68
P	Axial Load (working stress check)	kips	80	148
M	Moment (working stress check)	kip-in.	943	1240
P	for strength check		157	255
M	for strength check		2218	4032
A	cross sectional area	in.	12.60	20.00
S_x	section modulus	in3	62.70	103.00
Z_x	plastic modulus	in3	69.60	115.00
r_x	radius of gyration	in.	5.82	6.01
r_y	radius of gyration	in.	1.89	2.46
I_x	moment of intertia	in4	428	723
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.21	1.87
K_x			1.47	1.43
L unbraced length		in.	119	119
(KL/r)x			30.06	28.31
(KL/r)y			62.96	48.37
fa		ksi	6.35	7.40
fbx		ksi	15.04	12.04
Fa		ksi	22.21	24.60
Fbx		ksi	30.00	33.00
F'ex		ksi	165.30	186.27
fa/Fa			0.29	0.30
fbx/Fbx			0.50	0.36
Cmx			0.85	0.85
Cmx * fbx/(1 - fa/F'ex)/Fbx			0.44	0.32
fa/Fa + Cmx * fbx/(1 - fa/F'ex)/Fbx			0.73	0.62
fa/0.6F_y + fbx/Fbx			0.71	0.61
Strength Approach:				
Pcr		kips	475.64	836.47
P_y		kips	630.00	1000.00
P_e		kips	3991.96	7140.21
M_p		kips-in	3480.00	5750.00
Mm/M_p			0.93	1.00
P/Pcr + Cmx * M/(1 - P/Pe)/Mm			0.94	0.92
P/P_y + M/1.18/M_p			0.79	0.85

Defection Check:

Table B.3-4 shows the allowable drift ratio and the drift ratios obtained from the computer analysis, using the 1988 UBC, Chapter 23, Section 2312-h criteria.

Allowable Drift Ratio = min ($0.04/R_w$, 0.005) = 0.0033

Special requirement of UBC-88

1. Using the UBC-88, Section 2722 (d), the following criteria must be satisfied for columns.

   Exterior column only: (Since E=0 for interior column)

   $$P = 1.0\ DL + 0.7\ LL + 4.5\ E \qquad < 1.7\ F_a\ A$$

   $$P = 1.0 \times 67 + 0.7 \times 23 + 4.5 \times 17.5 = 162 \qquad < 475 \text{ kips OK}$$

2. Strength Ratio of Columns to Girder; Using the UBC - 88, Section 2722 (f) 5,

$$\frac{\Sigma Z_c\,(F_{y_c} - F_a)}{\Sigma Z_b\, F_{by}} > 1.0 \qquad \text{UBC(22–3)}$$

Exterior column f_a = 6.35 ksi
Z_c = 69.6 qu. in
Z_b = 112 qu. in

$$\frac{2(69.6)(50 - 6.35)}{112 \times 36} = 1.51 > 1.0 \quad \text{OK}$$

Interior column f_a = 7.4 ksi
Z_c = 115 ksi and Z_b = 112 ksi

$$\frac{2(115)(50 - 7.4)}{2(112)(36)} = 1.22 > 1.0 \quad \text{OK}$$

Table B.3-4, Drift Check, Transverse Direction

Level	Deflection from stress calc. (d)	Deflection (d/1.17)	Drift inches	Drift Ratio	Allowable Drift Ratio
R	0.947"	0.811"	0.245	0.0019	0.0033
3	0.660"	0.566"	0.318	0.0024	0.0033
2	0.289"	0.248"	0.248	0.0019	0.0033

B.4 DESIGN USING THE NAVY'S TECHNICAL MANUAL PROCEDURE

B.4.1 Site Seismicity

Two levels of earthquake motion are considered. At the first level, the structure is designed to remain elastic, or "near elastic", and the second level the structure responses inelastically but remains functional. The site response spectra are developed in accordance with the procedure of TM 5-809-10-1/NAVFAC P-355.1/AFM 88-3, Chapter 13, Section A, Technical Manual, "Seismic Design Guidelines for Essential Buildings", referred to herein as the Technical Manual. Assuming the building is located in seismic zone 4, (UBC Zoning), and following the procedure of the Technical Manual, Chapter 3, Section III, Figures 3-40 and 3-42, and Table 3-4, Aa and Av values for earthquakes EQ-I and EQ-II are obtained and are shown in Table B.4-1.

Assume also that the soil underlying the building is type S2, i.e. S_i=1.2 (The Technical Manual, Table 3-6). For structural steel system the damping values and the damping adjustment factors corresponding to EQ-I and EQ-II are obtained and are shown in Table B.4-2 (as per the Technical Manual, Tables 3-7 and 4-1).

Response spectra obtained from EQ-II represents a more severe earthquake and then the structure designed to these criteria should be checked primarily to satisfy drift limitations. The structure designed to a base shear obtained from EQ-I spectra should be checked using the strength approach since EQ-I represents first yield of the structure. In other words, while the structure designed for UBC criteria is checked using working stress method, the structure designed for EQ-I and EQ-II is checked using the ultimate stress method.

The governing equations for EQ-I and EQ-II response spectra are (The Technical Manual, Eq's. 3-27 and 3-28):

For T < 4.0 sec.
$S_a = 1.22 \times A_v \times S_i \times$ Damp. Adj. Factor/T (g)
but $< 2.5 \times$ Damp. Adj. Factor $\times A_a$ (g)

It follows that

EQ-I: $S_a = 1.22 \times 0.20 \times 1.2 \times 1.17 / T = .3426/T$ (g)
but $S_a < 0.585$ g

EQ-II: $S_a = 1.22 \times 0.45 \times 1.2 \times 0.90 / T = .5929/T$ (g)
but $S_a < 1.0125$ g
$(EQ\text{–}II)_{max}/(EQ\text{–}I)_{max} = 1.73$

Figure B.4-1 shows response spectra for EQ-I and EQ-II.

B.4.2 Response Spectra Analysis / Transverse Direction

B.4.2.1 General

The base shear coefficient and the members sizes obtained using UBC-85 are assumed to be the initial trial design. The EQ-I design spectra is compared to the static load coefficient (ZICS)

T, building period	S_a(g)	ZICS	S_a/ZICS
0.67 sec.	0.511	0.146	3.5

This ratio is greater than 1.7 (this is the load factor necessary to bring up UBC base shear to the ultimate load criteria), therefore, the structure designed for UBC-85 had to be modified for higher lateral force level. The member sizes that were obtained for use in response spectra analysis are shown in Figure B.4-2.

For dynamic analysis the transverse frames on lines 1, 4, and 7 are identical. Forces at the floors are distributed to frames in proportion to their relative rigidity. While there is no "calculated" torsion in the building, an "accidental" torsional shear is distributed to perimeter frames in both directions. The computer models that were developed for frame 4, is shown in Figure B.4-2. One third of the mass at each floor is carred by frame 4.

B.4.2.2 Natural Frequencies and Mode Shapes Analysis

Using the computer program ETABS the natural periods (T_m) and mode shapes (Φ_{xm}) are obtained in Table B.4-3. Table B.4-4 shows the modal analysis, i.e. modal story participation factors (PF_{xm}) and modal base shear participation factors (α_m), using the Technical Manual procedure, equations 4-1 and 4-2:

$$PF_{xm} = (\Sigma M_x \Phi_{xm} / \Sigma M_x \Phi_{xm}^2) \Phi_{xm} \tag{4-1}$$

$$\alpha_m = (\Sigma M_x \Phi_{xm})^2 / (\Sigma M_x \Sigma M_x \Phi_{xm}^2) \tag{4-2}$$

B.4.2.3 EQ-I Response

Table B.4-5 shows the modal base shears ($V_m = \alpha_m S_{am} W$, equation 4-4). Table B.4-6 shows modal story lateral forces (F_{xm}, The Technical Manual, eq. 4-3), modal shears and moments, and modal deflections and drifts (δ_{xm}, D_{xm}, The Technical Manual, eq. 4-5). Then the combinations of the modal values are obtained by taking the Square-Root-of-the-Sum-of-the-Squares (SRSS) of the values of all three modes.

B.4.3.2.4 EQ-II Response

The structural response to EQ-II is in the nonlinear range, therefore the period is assumed to lengthen by $\sqrt{x}$, where x represents the Inelastic Demand Ratio, IDR, of the critical elements in the frame (The Technical Manual, Table 4-2). In this example the x value is 1.25 for columns in a steel moment-resistant frame in essential facility. The same mode shapes obtained for EQ-I analysis is assumed to be valid for EQ-II analysis.

Table B.4-1, Aa & Av Values for EQ-I and EQ-II

Prob. of Exceedance	ATC-3-06 10% in 50 years	EQ-I 50% in 50 years	EQ II 10% in 100 years
Aa	0.40 g	0.20 g	0.45 g
Av	0.40 g	0.20 g	0.45 g

Table B.4-2, Damping Adjustment Factors

	Damping	Damp.Adj. Factor
EQ-I	3 %	1.17
EQ-II	7 %	0.90

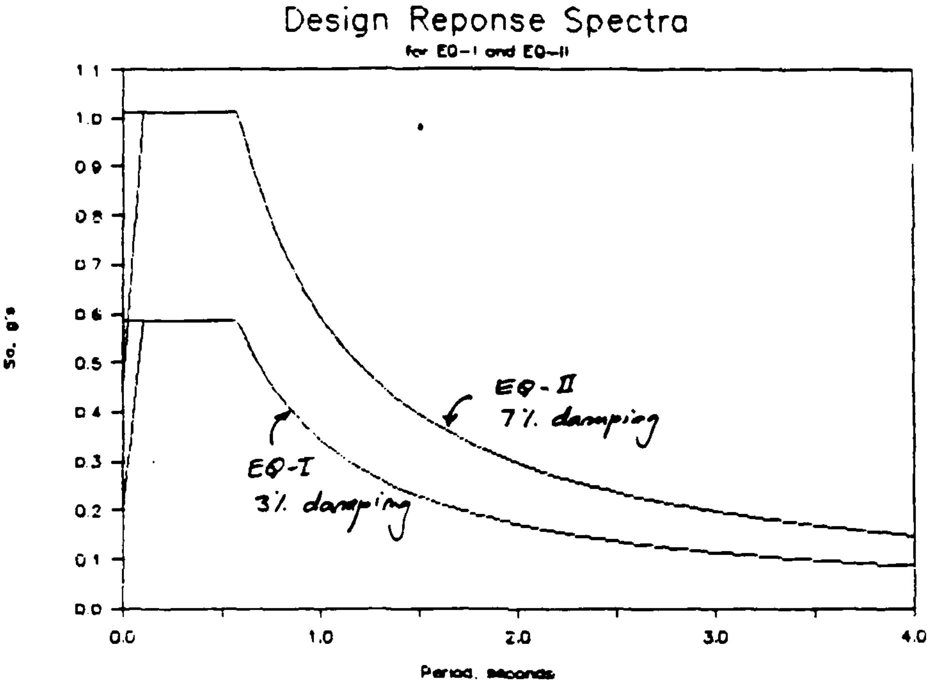


Figure B.4-1, Design Response Spectra for EQ-I and EQ-II

Table B.4-3, The Periods and the Mode Shapes for the Transverse Direction

Level	Mass $\frac{K.\ sec^2}{ft.}$	Mode 1		Mode 2		Mode 3	
		Φ_{x1}	T_1	Φ_{x2}	T_2	Φ_{x3}	T_3
R	3.88	1.2346	.457	1.1648	.167	.4614	.094
3	7.24	.8404	sec	-.5816	sec	-.7806	sec
2	7.24	.3663		-.7695		.9665	
	$\sum$ 18.36						

Table B.4-4, Modal Participation Factors, EQ-I

	Mode 1		Mode 2		Mode 3	
	$M_x\Phi_{x1}$	$M_x\Phi_{x1}^2$	$M_x\Phi_{x2}$	$M_x\Phi_{x2}^2$	$M_x\Phi_{x3}$	$M_x\Phi_{x3}^2$
	4.790	5.914	4.519	5.264	1.790	0.826
	6.085	5.114	-4.211	2.449	-5.651	4.411
	2.652	0.972	-5.571	4.286	6.997	6.763
$\sum$	13.527	12.000	-5.262	12.000	3.136	12.000
PF_{rm}	1.392		-0.511		0.121	
PF_{3m}	0.947		0.255		-0.204	
PF_{2m}	0.413		0.337		0.253	
α_m	0.831		0.126		0.045	

Table B.4-5, Modal Base Shears, EQ-I

		Mode 1	Mode 2	Mode 3
T_m	sec	0.457	0.167	0.094
S_{am}	g's	0.585	0.585	0.585
$\alpha_m S_{am}$		0.486	0.074	0.026
$V_m = \alpha_m S_{am} (\sum mg)$	Kips	287.3	43.5	15.4

Table B.4-6, EQ-I Response

Mode	Level	PF_{xm}	F_{xm} K.	V_{xm} K.	$DOTM_{xm}$ Kip-ft	OTM_{xm} Kip-ft	δ_{xm} in.	D_{xm} in.
1	R 3 2	1.392 .947 .413	101.7 129.2 56.3	101.7 230.9 287.3	1242 2540 3065	000 1242 3782 6847	1.668 1.135 0.495	.532 .640 .495
2	R 3 2	-.511 .255 .337	-37.3 34.8 46.0	-37.3 2.5 43.5	-456 28 464	000 -456 -484 -20	-.081 .041 .054	.122 .013 .054
3	R 3 2	.121 -.204 .253	8.8 -27.8 34.5	8.8 -19.0 15.4	108 -209 165	000 108 -101 63	.006 -.010 .013	.016 .036 .013
S R S S	R 3 2		108.7 136.7 80.5	108.7 231.7 290.9	1327 2550 3104	000 1327 3814 6848	1.670 1.136 .498	.547 .641 .498

Table B.4-7, Modal Base Shears, EQ-II

		Mode 1	Mode 2	Mode 3
$T_m\sqrt{1.25}$	sec	0.511	0.186	0.105
S_{am}	g's	1.013	1.013	1.013
$\alpha_m S_{am}$		0.841	0.127	0.045
$V_m = \alpha_m S_{am} (\sum mg)$	Kips	497.2	75.2	26.7

Table B.4-8, EQ-II Response

Mode	Level	PF_{xm}	F_{xm} K.	V_{xm} K.	$DOTM_{xm}$ Kip-ft	OTM_{xm} Kip-ft	δ_{xm} in.	D_{xm} in.
	R	1.392	176.1	176.1	2150	000	3.608	1.152
1	3	.947	223.6	399.7	4397	2150	2.456	1.385
	2	.413	97.5	497.2	5305	6546	1.071	1.071
						11851		
	R	-.511	-64.6	-64.6	-789	000	-.176	.264
2	3	.255	60.2	-4.4	-49	-789	.088	.028
	2	.337	79.6	75.2	803	-837	.116	.116
						-35		
	R	.121	15.3	15.3	186	000	.013	.036
3	3	-.204	-48.2	-32.9	-362	186	-.022	.050
	2	.253	59.6	26.7	285	-362	.028	.028
						285		
S	R		188.2	188.2	2298	000	3.612	1.182
R	3		236.5	401.1	4412	2298	2.458	1.387
S	2		139.3	503.5	5373	6602	1.077	1.077
S						11851		

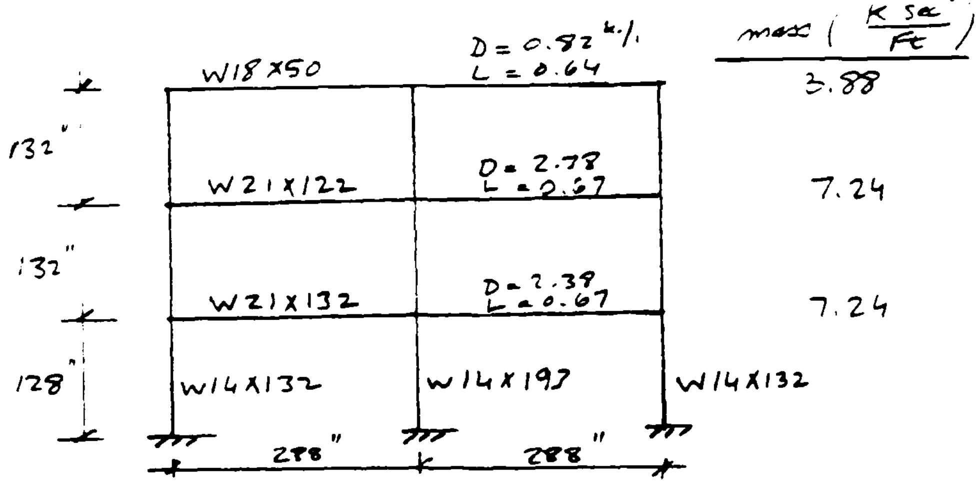
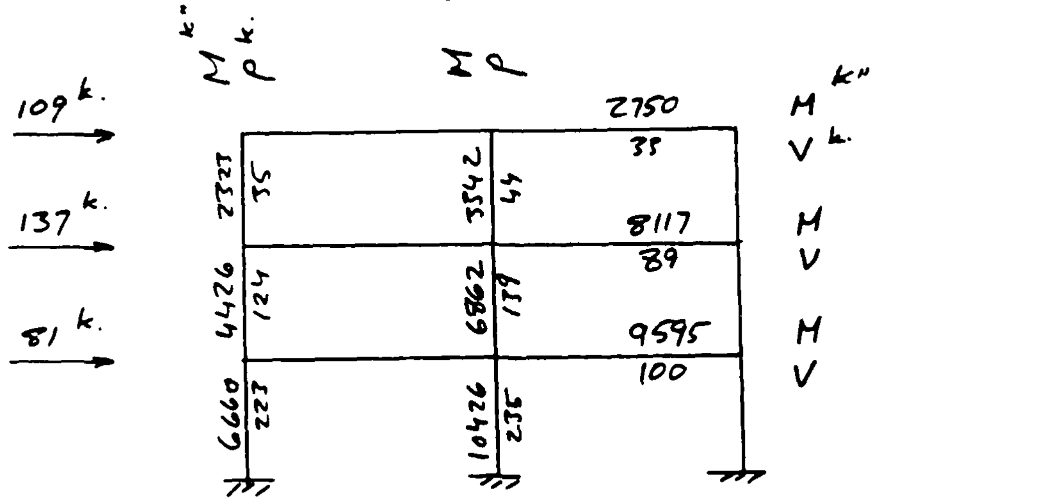
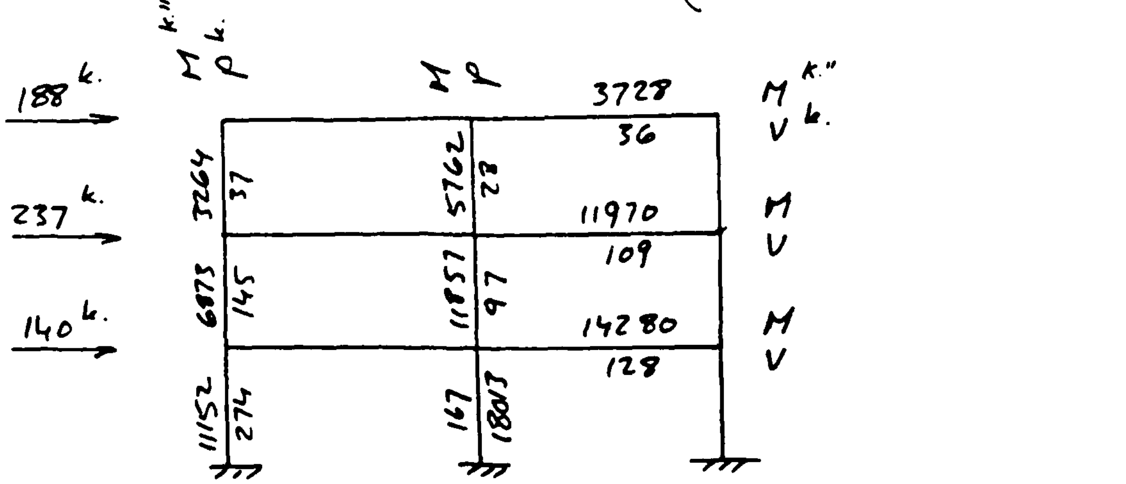


Figure B.4-2



EQ-I : 1.2D + L ± E (the Tech. Manual Eq. 4-6)



EQ-II : 1.0D + .25L ± E (the Tech. Manual Eq. 4-9)

Figure B.4-3

Table B.4-7 shows the calculations performed to obtain the modal base shear (V_m, The Technical Manual, eq. 4-4). Then Table B.4-8 shows modal story lateral forces (F_{xm}, The Technical Manual, eq. 4-5). Then SRSS rule is applied to obtain modal combination values.

B.4.3 Checking Transverse Direction for EQ-I & EQ-II

The computer results for the static analysis for frame 4 are shown in Figure B.4-3. These results are obtained using the static lateral loads obtained from response spectra analysis for EQ-I and EQ-II.

The stress check for beams is performed using the strength approach, i.e. the ultimate strength method,, where the moment demand is compared against the ultimate moment capacity of the beams. These calculations are shown in Table B.4-9. For the DMRSF in the transverse direction, 20% of the beams at any story are allowed to exceed the flexural strength requirements by up to 25% for EQ-I loading, (The Technical Manual, Para. 4-3e(1)). While for EQ-II loading, the Inelastic Demand Ratio, IDR, (demand/capacity) is allowed to reach 2.0, (The Technical Manual, Table 4-2).

The stress check for columns is performed using also the ultimate strength approach. This was done in the following ways:

EQ-I
****

criteria: 1. Strong columns weak beams frame. Assume plastic hinge formation in all beams while columns remain elastic. See figure B.2-2. Use equations 2.4-2 and 2.4-3, AISC Spec. 8th edition.

2. Check the computer output using equations 1.6-1a and 1.6-1b, AISC Spec. 8th edition, and allow an increase of 1.7 of the allowable stresses.

Level	Beam	M_p kip-in	$V=2M_p/L$ kips
R	W18 × 50	3728	26
3	W21 × 122	11970	83
2	W21 × 132	14280	99

			208

1. Exterior column;

$M = 0.55\ M_p\ (\text{beam}) = 0.55 \times 14280 = 7854$ kip-in
$P = 1.3\ (D+L) + 2M_p/L = 1.3\ (93) + 208 = 329$ kips

Interior column;

$M = 1.0\ M_p(\text{beam}) = 1.0 \times 14280 = 14280$ kip-in
$P = 1.3\ (D+L) + 2M_p/L = 1.3\ (154 + 50) = 265$ kips

Use equations: AISC 8th edition Spec. (2.4-2 and 2.4-3).

$$\frac{P}{P_{cr}} + \frac{C_{mx}\,M}{(1 - \frac{P}{P_c})\,M_m} < 1 \quad , \quad \frac{P}{P_y} + \frac{M}{1.18\,M_p} < 1$$

2. From computer output:

Using load combination of (1.2 DL + LL + E), according to The Technical Manual, equation 4-6

Design forces:	exterior column	M	=	6660	kip-in
		P	=	223	kips
	interior column	M	=	10426	kip-in
		P	=	235	kips

Use equations: AISC 8th edition Spec. (1.6-1a and 1.6-1b)

$$\frac{f_a}{1.7\,F_a} + \frac{C_{mx}\,f_{bx}}{(1 - \frac{f_a}{1.7\,F'_{ex}})\,1.7\,F_{bx}} < 1 \,, \quad \frac{f_a}{0.6\,(1.7\,F_y)} + \frac{f_{bx}}{1.7\,F_{bx}} < 1$$

Table B.4-10 shows columns check calculations for EQ-I.

Table B.4-9, Beams Check

Level	Size	Zx qu.in	EQ-I Md Kip-in	EQ-I Mc Kip-in	EQ-I Md/Mc	EQ-II Md Kip-in	EQ-II Mc Kip-in	EQ-II Md/Mc	IDR
R	W18x50	101	2750	3636	0.76	3728	3636	1.03	2.0
3	W21x122	307	8117	11052	0.73	11970	11052	1.08	2.0
2	W18x132	333	9595	11988	0.80	14280	11988	1.19	2.0

Table B.4-10, Columns Check for EQ-I

	Description		Exterior Column Level 1	Interior Column Level 1
	Column section	GR 50	W14 x 132	W14 x 193
P	Axial Load (Technical Manual)	kips	223	235
M	Moment (Technical Manual)	kip-in.	6660	10426
P	for strength check (AISC)		329	265
M	for strength check (AISC)		7854	14280
A	cross sectional area	in.	38.80	56.80
S_x	section modulus	in3	209.00	310.00
Z_x	plastic modulus	in3	234.00	355.00
r_x	radius of gyration	in.	6.28	6.50
r_y	radius of gyration	in.	3.76	4.05
I_x	moment of intertia	in4	1530	2400
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.19	1.72
K_x			1.48	1.41
L unbraced length		in.	119	119
(KL/r)x			28.04	25.81
(KL/r)y			31.65	29.38
fa		ksi	5.75	4.14
fbx		ksi	31.87	33.63
1.7*Fa		ksi	45.81	46.29
1.7*Fbx		ksi	56.10	56.10
1.7*F'ex		ksi	322.78	380.97
fa /1.7Fa			0.13	0.09
fbx/1.7Fbx			0.57	0.60
Cmx			0.85	0.85
Cmx*fbx/(1 - fa/1.7/F'ex)/1.7/Fbx			0.49	0.52
fa/1.7/Fa + Cmx*fbx/(1 - fa/1.7/F'ex)			0.62	0.60
fa/1.7/0.6F_y + fbx/1.7/Fbx			0.68	0.68
Strength Approach:				
Pcr		kips	1777.40	2629.21
P_y		kips	1940.00	2840.00
P_e		kips	14119.89	24397.24
M_p		kips-in	11700.00	17750.00
Mm/M_p			1.00	1.00
P/Pcr + Cmx*M/(1 - P/Pe)/Mm			0.77	0.79
P/P_y + M/1.18/M_p			0.74	0.78

EQ-II
****

For column check for EQ-II loading, The Technical Manual procedure, Figure 4-2, is followed using the computer output for a load combination of (1.0 DL + 0.25 LL + E), according to equation 4-7 of The Technical Manual.

Design forces:	exterior column	M	=	11152	Kip-in
		P	=	274	Kips
	interior column	M	=	18013	Kip-in
		P	=	167	Kips

The equations of Figure 4-2 of The Technical Manual are:

(1) $\frac{M_x}{M_{P_{cx}}} < \text{IDR}$, (2) $\frac{C_{mx} M_x}{M_{U_{cx}}} < \text{IDR}$ where

$$M_{P_{cx}} = 1.18\, M_{P_x} \left(1 - \frac{P}{P_y}\right) \quad \text{and} \quad M_{u_{cx}} = M_{P_x} \left(1 - \frac{P}{P_{cr}}\right)\left(1 - \frac{P}{P_{cx}}\right)$$

Table B.4-11 shows columns check calculations for EQ-II where the Inelastic Demand Ratio IDR is allowed to reach 1.25, (The Technical Manual, Table 4-2).

Table B.4-12 shows drift check for EQ-I and EQ-II following The Technical Manual drift ratio limits, para. 4-3(e) 7(a), and para. 4-4(e) 2(a).

Table B.4-11, Columns Check for EQ-II

Col. GR50	P_d K.	M_d K-in	M_{p_x} K-in	$M_{p_{cx}}$ K-in	$M_{u_{cx}}$ K-in	eq.1	eq.2	IDR
W14x132	272	11152	11700	12140	9704	0.92	0.98	1.25
W14x193	176	18013	17750	19713	16510	0.91	0.91	1.25

Table B.4-12, Drifts Check

Level	EQ-I			EQ-II		
	Drift (in.)	Drift Ratio	Allowable Drift Ratio	Drift (in.)	Drift Ratio	Allowable Drift Ratio
R	0.547	0.0041	0.005	1.182	0.0089	0.010
3	0.641	0.0049	0.005	1.387	0.0105	0.010
2	0.498	0.0039	0.005	1.077	0.0084	0.010

APPENDIX C

BUILDING ON BASE ISOLATORS

C.1 Description Building

The same three-story essential building that was considered in Appendix B is also considered here. The lateral seismic resistance is provided by transverse ductile moment-resisting steel frames (DMRSF) and longitudinal steel braced frames (BSF). The building is located in UBC Seismic Zone No. 4 and the soil profile of the site consists of a dense soil where the depth exceeds 200 feet. Only the transverse direction is considered here.

C.2 Site Seismicity

The Navy's Technical Manual procedure is used to define site seismicity. Two levels of earthquake motion are considered. The EQ-I earthquake ground motion has a 50% chance of being exceeded in a 50 year time period. The second design earthquake EQ-II, the most probable maximum earthquake, has approximately a 10% chance of being exceeded in a 100 year time period.

The site response spectra that are used here are the same as those developed in Appendix B for the design of the fixed base building. For structural steel (DMRSF) the damping values and the adjustment factors corresponding to EQ-I and EQ-II are obtained in table C.1.

Table C.1

	Damping	Damping Factor
EQ-I	3 %	1.17
EQ-II	7 %	0.90

C.3 Analysis 1

Analysis 1 is the analysis used to develop the design and it is consistent with the Technical Manual procedure. A 2-D response spectra analysis is performed here. In order to obtain the initial trial design for the frame and the isolators, the portal method is used to size the members using the Preliminary Design provisions defined in the design criteria, (Equations C-1 thru C-5). Then successive iterations were made to insure that the isolated building period does not exceed 3.0 seconds and is approximately four times the fixed base building period. The dead loads were increased by 10% to account for the effects of vertical accelerations on beams and columns members.

Isolation modeling for the ETABS program was done by inserting a "dummy" story level below the base level of the frame model, and assign the column properties to reflect the isolators' stiffness:

Axial Area	A	=	$K_a H/E$
Shear Area	A_j	=	0
Moment of Inertia	I	=	$\dfrac{k_{eff} H^3}{3E}$

where H is the dummy story height, E is Young modulus, K_a is the isolator axial stiffness, and k_{eff} is the isolator effective lateral stiffness. This isolator effective lateral stiffness was found to be 3.655 kips/inch.

Using the computer program ETABS the following periods and mode shapes are obtained. (Table C.2).

Periods and Mode Shapes
Table C.2

Level	Mass $\frac{k.sec2}{ft.}$	Mode 1		Mode 2		Mode 3	
		ϕx1	T_1	ϕx2	T_2	ϕx3	T_3
R	3.88	0.8241	2.85	1.3514	0.278	0.7664	0.132
2	7.24	0.8131	sec	0.0688	sec	-0.9958	sec
1	7.24	0.7952		-0.8210		0.5926	
B	0.00	0.7545		-1.0819		1.2823	
Modal Participation Factors		1.2368		-0.0168		0.0045	

C.3.1 EQ-I Response

The structural response to EQ-I is in a near elastic range, therefore, the percent damping is 3. Table C.3 shows the modal base shear (V_m), the total lateral force corresponding to mode m.

Table C.3 also shows modal story lateral forces and modal cumulative story shears. Then the combinations of the modal values are obtained by taking the Square-Root-of-the-Sum-of-the-Squares (SRSS) of the values of all three modes.

C.3.2 EQ-II Response

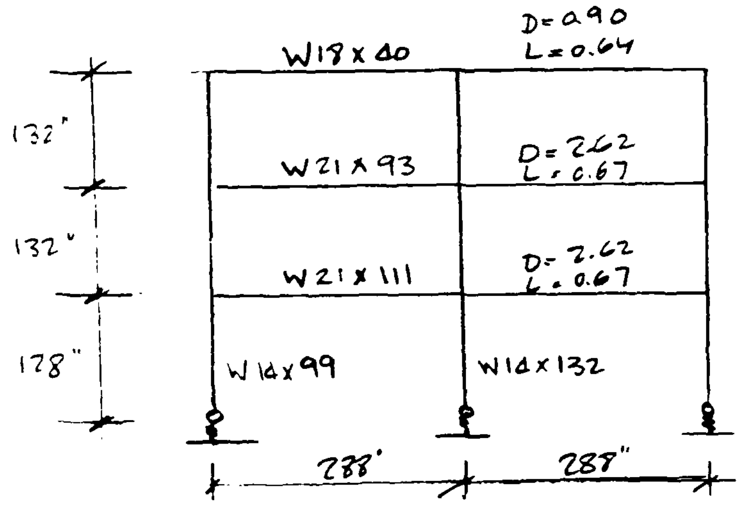
The structural response to EQ-II is assumed to have the same periods and mode shapes as in the linear EQ-I response. Table C.4 shows the modal story lateral forces and the modal cumulative story shears. Then SRSS rule is applied to obtain modal combination values.

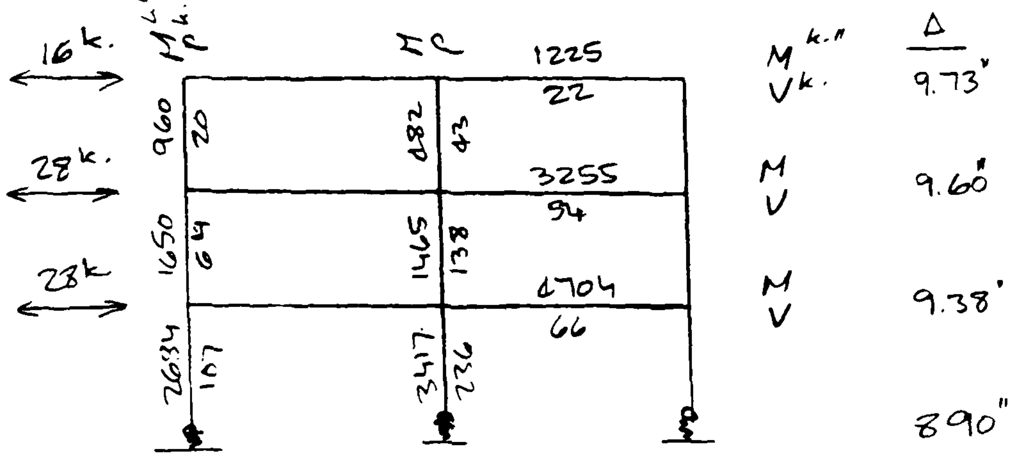
Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	15.28	-1.66	.25	15.37
2	28.13	-.16	-.62	28.14
1	27.51	1.88	.37	27.58
B	.00	.00	.00	.00
Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	15.28	-1.66	.25	15.37
2	43.41	-1.82	-.36	43.45
1	70.92	.06	.00	70.92
B	70.92	.06	.00	70.92

EQ-I Response Spectra Analysis
Table C.3

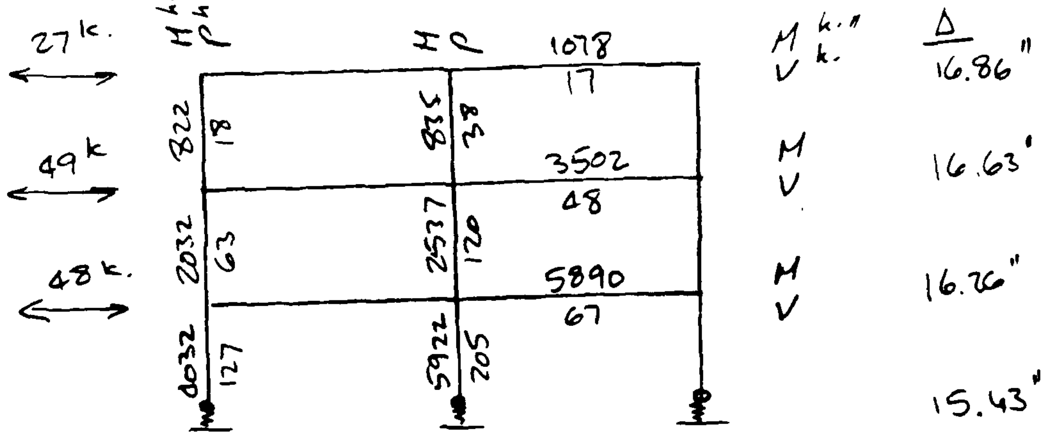
Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	26.48	-2.88	.44	26.64
2	48.76	-.27	-1.07	48.77
1	47.68	3.26	.63	47.80
B	.00	.00	.00	.00
Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	26.48	-2.88	.44	26.64
2	75.24	-3.15	-.63	75.31
1	122.92	.11	.01	122.92
B	122.92	.11	.01	122.92

EQ-II Response Spectra Analysis
Table C.4





EQ-I : 1.2D + L ± E



EQ-II = 1.0D + 0.25L ± E

Figure C.1

C.3.3 Checking Member Sizes for EQ-I & EQ-II

The computer results for the response spectra analysis for frame 4 are shown in Figure C.1.

C.3.3.1 Checking Beams

The following table shows beams check in accordance with the procedure of The Technical Manual.

			EQ-I			EQ-II			
Level	Size	Z_x	M_d	M_c	M_d/M_c	M_d	M_c	M_d/M_c	IDR
R	W18x40	78.4	1255	2822	0.43	1078	1822	0.38	2.0
2	W21x93	221	3255	7956	0.41	3502	7956	0.44	2.0
1	W21x111	279	4704	10044	0.47	5890	10044	0.59	2.0

C.3.3.2 Checking Columns

Table C.5 shows column check for EQ-I and EQ-II responses.

EQ-I
****

criteria:
1. strong columns weak beams frames assume plastic hinge formation in all beams
2. apply The Technical Manual Procedure

LEVEL	BEAM	M_p kips.in	$V=2M_p/L$ kips
R	W18x40	2822	20 kips
2	W21x93	7956	55 kips
1	W21x111	10044	70 kips
			---------
			145 kips

1. Exterior Column;

$M = 0.55\ M_p\ \text{(beam)} = 0.55 \times 10044 = 5525$ kips.in

$P = 1.3\ (D + L) + 2M_p/L = 1.3\ (97) + 145 = 271$ kips

Interior Column;

M	=	1.0 M_p (beam)	=	1.0 x 10044	=	10044	kips.in
P	=	1.3 (D + L) + $2M_p/L$	=	1.3 (170 + 50)	=	286	kips

2. From computer output
   (1.2DL + LL + E)

Design Forces:	exterior column	M	=	2684	kips.in
		P	=	107	kips
	interior column	M	=	3417	kips.in
		P	=	236	kips

EQ-II
****

-From computer output
(1.0DL + 0.25LL + E)

Design Forces:	exterior column	M	=	4032	kips.in
		P	=	127	kips
	interior column	M	=	5922	kips.in
		P	=	205	kips

checking the columns are performed in Table C.5.

C.3.3.3 Checking Drifts

The following table shows drift ratios check for EQ-I and EQ-II.

	EQ-I			EQ-II		
Level	drift		allowable drift	drift		allowable drift
R	0.13	0.0001	0.005	0.23	0.0017	0.010
2	0.22	0.0017	0.005	0.37	0.0028	0.010
1	0.48	0.0038	0.005	0.83	0.0065	0.010

Beam-Column Check for Analysis - 1
Table C.5

Description			Exterior Column Level 1	Interior Column Level 1
	Column section	GR 50	W14 × 99	W14 × 132
P	for EQ-I (Strength Check)	kips	271	286
M	for EQ-I (Strength Check)	kips.in	5,525	10,044
P	for EQ-II (Navy Check)	kips	127	205
M	for EQ-II (Navy Check)	kips.in.	4,032	5,922
A	cross sectional area	in.	29.10	38.80
S_x	section modulus	in3	157.00	209.00
Z_x	plastic modulus	in3	173.00	234.00
r_x	radius of gyration	in.	6.17	6.28
r_y	radius of gyration	in.	3.71	3.76
I_x	moment of intertia	in4	1110	1530
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.21	1.87
K_x			1.47	1.43
L unbraced length		in.	119	119
(KL/r)x			28.35	27.10
(KL/r)y			32.08	31.65
Fa		ksi	26.89	26.95
F'ex		ksi	185.78	203.38
Cmx			0.85	0.85
Strength Approach AISC for EQ-I:				
Pcr		kips	1,330	1,777
P_y		kips	1,455	1,940
P_e		kips	10,362	15,125
M_p		kips-in	8,650	11,700
Mm/M_p			1.00	1.00
P/Pcr + Cmx * M/(1 - P/Pe)/Mm			0.76	0.90
$P/P_y + M/1.18/M_p$			0.73	0.87
Navy Criteria for EQ-II:				
$M_{pcx} = 1.18\ M_{px}\ (1 - P/P_y)$			9,316	12,347
$M_{ucx} = M_{px}\ (1 - P/P_{cr})\ (1 - P/P_{ex})$			7,728	10,210
$M_x\ /\ M_{pcx}$			0.43	0.48
Cmx M_x/M_{ucx}			0.44	0.49

C.3.3.4 Load-Deflection Inelastic Analysis, Post-Yielding Capacity

Since a response spectra analysis is used for EQ-II earthquake, a load deflection inelastic analysis is required. A check for the ultimate capacity of the Frame 4 on base isolators is performed. The force-displacement curve is obtained by the following procedure:

1. Define the force that causes the first major yield in the frame. Then calculate the associated roof displacement, these force-displacement value plots the point A on the force-displacement curve.

   The forces to produce yielding of a component are obtained by the combinations:

   1.0 E + 0.8 DL

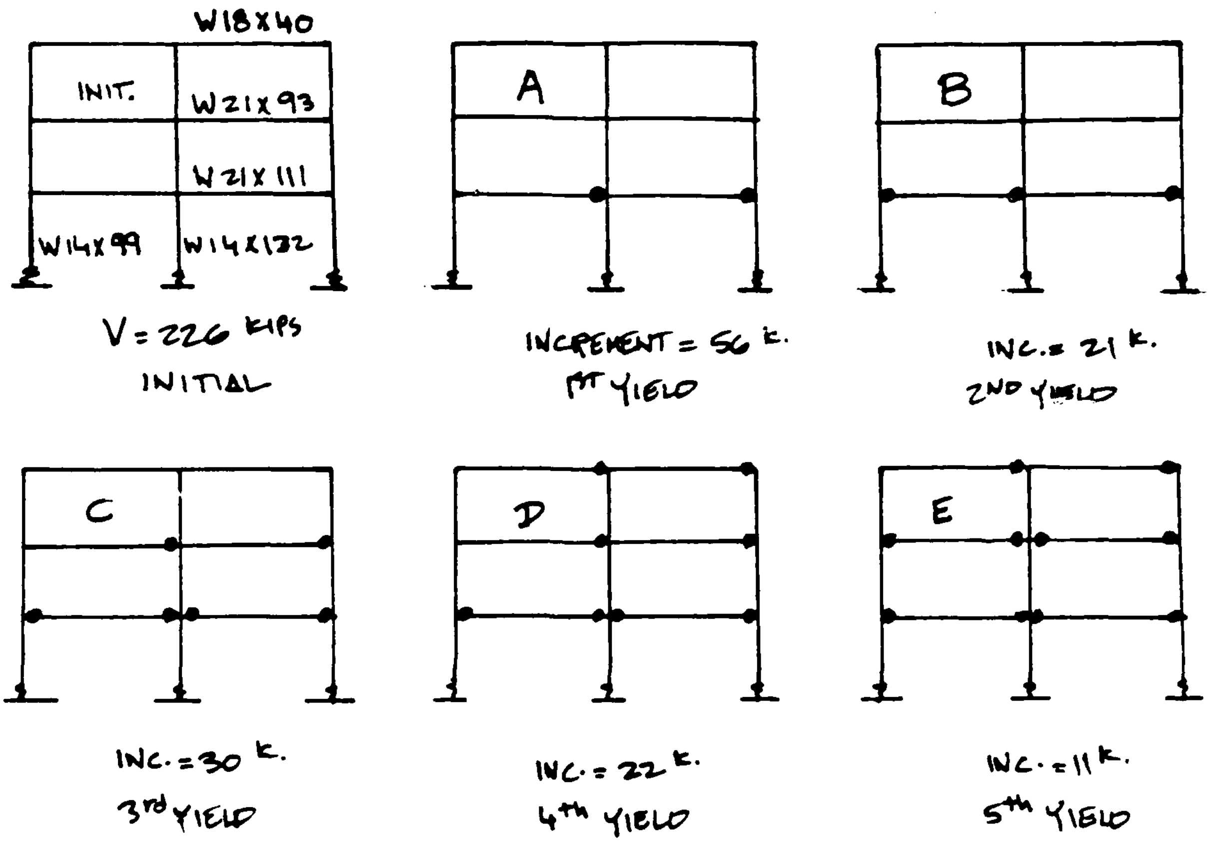
   1.0 E - 1.2 DL - 1.0. LL

2. Determine the first post-yield segment of the curve by freezing the frame at the point of initial major yield, and calculate the incremental force that causes the second yield of the frame. Then calculate the incremental roof displacement. The total force-displacement value plots the point B on the curve.

3. Determine sequential post-yield segments on the curve by repeating the same procedure above.

4. The procedure is repeated until a failure mechanism, or instability occurs.

5. In this analysis, the increase of the total base shear force is distributed to the stories by a dynamic analysis, i.e. response spectra analyses. After each major yield, and in order to determine the next post yielding segment, a dynamic analysis is performed on the frame to calculate the new periods and mode shapes. Then these new periods and mode shapes are used to calculate the percentage of base shear force that is taken by each story.



Limit State Analysis

Figure C.2

Figure C.2 shows the sequence of plastic hinges formations in the frame, and Tables C.6 show a comparison between the force-displacement results for the frame designed with fixed base and isolated base. Figure C.3 shows plots of the base shear-drift capacity curves and the base shear coefficient-drift capacity curves. And finally Table C.7 shows comparison between the weights of the frame members designed for the navy criteria for fixed base and isolated base buildings.

Fixed Base Design

Point	Base Shear kips	V/W	Roof Displ. in.	Roof Drift Ratio
Int.	1392	0.778	2.48	0.0063
A	1512	0.845	2.78	0.0070
B	1581	0.884	3.32	0.0084
C	1617	0.903	4.12	0.0104
D	1635	0.914	5.73	0.0145

Base Isolated Design

Point	Base Shear kips	V/W	Roof Displ. in.	Isolator Displ.	Roof Drift Ratio
Int.	678	0.38	30.95	28.33	0.0067
A	846	0.47	39.25	35.41	0.0098
B	909	0.51	42.5	38.05	0.0114
C	999	0.56	48.05	41.81	0.0169
D	1065	0.60	53.04	44.50	0.0218
E	1098	0.61	57.99	45.89	0.0309

Tables C.6

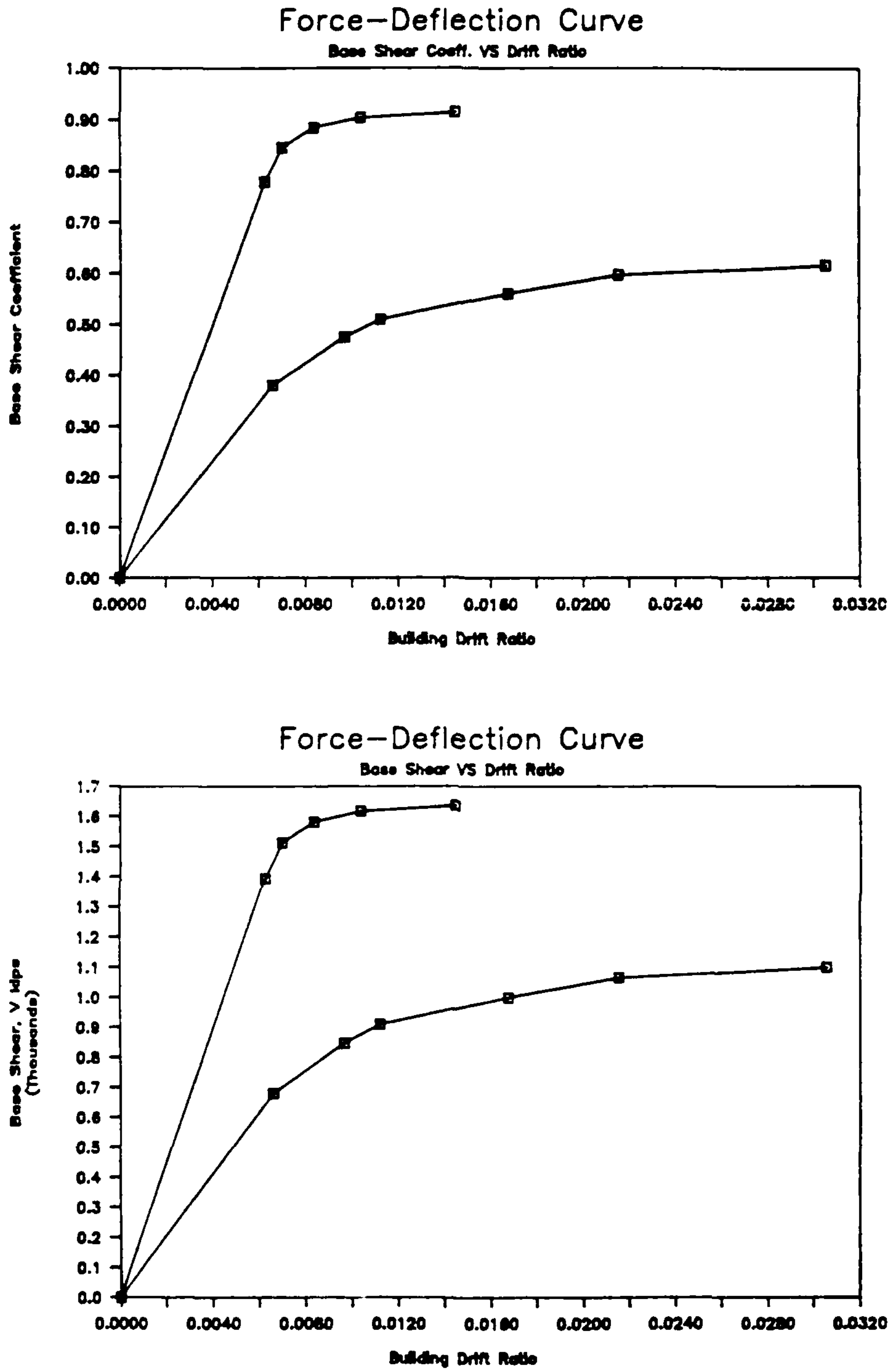


Figure C.3
Force-Deflection Curves for
Fixed-Base vs Base-Isolated Designs

Table C.7
Comparison Between Frame Total Weight

Base Isolated Design	Beams	:	11712	lb.	(GR36)
	Columns	:	10780	lb.	(GR50)
	Total	:	22492	lb.	
Fixed Base Design	Beams	:	14592	lb.	(GR36)
	Columns	:	14926	lb.	(GR50)
	Total	:	29520	lb.	
Percentage saving in steel weight is 25%.					

C.4 Analysis 2

Analysis 2 is a response spectra analysis that provides a lower bound safety net design. The structure designed in Analysis 1 is assumed to be fixed base and then subjected to EQ-I design earthquake. The Inelastic Demand Ratios of the beams and the columns must be less than 3. Table C.8 shows EQ-I response spectra analysis: the fixed base periods, mode shapes, and modal participation factors. Table C.9 shows maximum modal story forces and maximum modal cumulative story shears and the SRSS modal combination. Figure C.4 shows element forces for load combination 1.2D + 1.0L ± E.

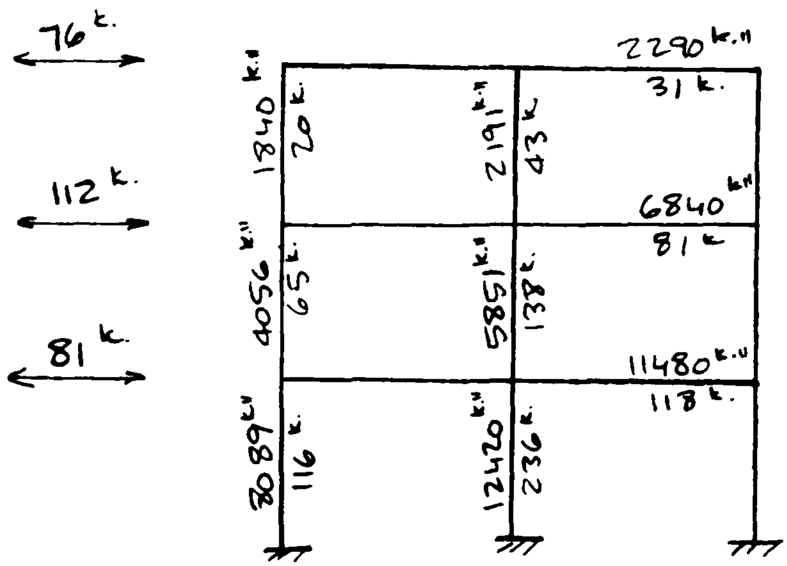
Table C.8
Analysis 2, Periods and Mode Shapes

Level	Mass $\frac{k.sec2}{ft.}$	Mode 1		Mode 2		Mode 3	
		ϕ_{x1}	T_1	ϕ_{x2}	T_2	ϕ_{x3}	T_3
R	3.88	1.0454	0.753 sec	1.2661	0.234 sec	0.6300	0.123 sec
2	7.24	0.8581		-0.2470		-0.9275	
1	7.24	0.5792		-0.8587		0.7646	
Modal Participation Factors		1.2052		-0.2577		0.1054	

Analysis 2, Response Spectra Analysis
Table C.9

Maximum Modal Story Forces (kips)				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	71.93	-23.85	4.86	75.93
2	110.17	8.68	-13.34	111.31
1	74.37	30.18	10.99	81.01

Maximum Modal Cumulative Story Shears				
LEVEL	MODE 1	MODE 2	MODE 3	SRSS
R	71.93	-23.85	4.86	75.93
2	182.09	-15.17	-8.48	182.92
1	256.46	15.01	2.51	256.91



EQ-I = 1.2D + L ± E
Analysis 2

Figure C.4

C.4.1 Inelastic Demand Ratios Check

The Inelastic Demand Ratios for beams or columns can not exceed 3.0. Tables C.10 and C.11 show the IDR's checks for the beams and column using the response spectra computer output and the Navy's Technical Manual design criteria.

Table C.10
Beams IDR Check for Analysis 2

Level	Beam	M_p kips.in	M_d kip.in	M_d/M_p	IDR
R	W18x40	2822	2290	0.81	3.0
2	W21x93	7956	6840	0.86	3.0
1	W21x111	10044	11480	1.15	3.0

Beam-Column IDR Check for Analysis - 2
Table C.11

	Description		Exterior Column Level 1	Interior Column Level 1
	Column section	GR 50	W14 × 99	W14 × 132
P	for analysis-2	kips	116	236
M	for analysis-2	kips.in.	8,089	12,420
A	cross sectional area	in.	29.10	38.80
S_x	section modulus	in3	157.00	209.00
Z_x	plastic modulus	in3	173.00	234.00
r_x	radius of gyration	in.	6.17	6.28
r_y	radius of gyration	in.	3.71	3.76
I_x	moment of intertia	in4	1110	1530
K_y			1.00	1.00
G_x bottom			1.00	1.00
G_x top			2.21	1.87
K_x			1.47	1.43
L unbraced length		in.	119	119
(KL/r)x			28.35	27.10
(KL/r)y			32.08	31.65
Fa		ksi	26.89	26.95
F'ex		ksi	185.78	203.38
Cmx		ksi	0.85	0.85
Navy Criteria:				
$M_{pcx} = 1.18\ M_{px}\ (1 - P/P_y)$			9,393	12,127
$M_{ucx} = M_{px}\ (1 - P/P_{cr})\ (1 - P/P_{ex})$			7,807	9,988
$M_x\ /\ M_{pcx}$			0.86	1.02
Cmx M_x/M_{ucx}			0.88	1.06